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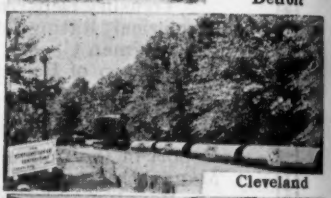
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ERRATUM

VOL. 24, NO. 7, JULY, 1932

Page 1049: Under Section D, the phrase "if at no time within 24 hours more than 10 percent gas is observed" should read "if at no time within 48 hours gas is observed".

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Vol. 24 SEPTEMBER, 1932 No. 9

COLON-AEROGENES TYPES OF BACTERIA AS CRITERIA OF FECAL POLLUTION

By Fred O. Tonney[1] and Ralph E. Noble[2]

With the recognition of the rôle of drinking water in the spread of intestinal disease, the demand arose for some test characteristic of fecal material, which could be applied to water supplies to detect dangerous fecal pollution.

Chemical methods were first used, including tests for chlorides as an inorganic constituent of animal excrement, for oxygen consumed as a rough measure of organic matter, and for free and albuminoid ammonia, nitrites and nitrates, as an index of nitrogenous matter and its disintegration products. Soon, however, it became evident that, since these chemical substances were common to all forms of organic matter as well as fecal material, their presence did not afford a dependable means of distinguishing safe from unsafe waters.

Meanwhile, attention turned to bacteriological methods, at first to find the direct causative agents of water-borne disease in water, such as the typhoid and paratyphoid group of bacteria, and later to detect some other organisms definitely associated with feces. The attempt to test water routinely for the causative pathogenic organisms was soon abandoned as impracticable, on account of their relatively

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small numbers and the complexity of the methods for their identification. It was recognized, however, that the colon group of organisms was by far the most predominant and constant of the bacterial constituents of intestinal discharges. Moreover, these organisms proved easier of detection than the organisms directly causing water-borne disease. Hence, in due course of time, the presence of the "colon group" of bacteria came to be accepted as a test for fecal matter and gradually displaced the less sensitive chemical tests in routine procedure. Then began the evolution of simpler routine tests for the detection of the colon group of organisms in water.

The first step in the earlier routine for detection of the colon group, was a fermentation test in dextrose broth (1). Later, dextrose gave way to lactose in the broth (2), and this has remained the standard method ever since. Upon the preliminary or "presumptive" fermentation test was built a rough method of estimating numbers of colon bacteria, based on inoculation of fractional protions of the sample into the liquid fermentation medium, and a system of "confirmatory" tests was also devised, tending to exclude other more common lactose fermenting bacteria.

Unfortunately, as we now know, the system of the standard routine procedure brings out prominently in the findings another family of organisms having certain fermentative and other properties in common with the colon group. This latter class of organisms, known as the aerogenes group, occurs also to a limited extent in feces, and for a time was also considered to be a natural inhabitant of the intestinal tract.

Other lactose fermentative media have been proposed from time to time, as improvements on the standard lactose broth. They contain mostly dyes or bile salts or both (3–9), and in principle act to inhibit the growth of other lactose fermenting bacteria to a greater degree than they inhibit the colon group. All these media, however, permit the growth of the Bact. aerogenes as well as the Bact. coli type, and the findings as reported are based on the combined presence of the two types, without respect to their relative numbers in the original samples.

The difficulties which these fermentative methods present in routine practice, are: (1) metabolic interference with the growth of the colon-aerogenes group by extraneous forms; (2) the large number of lactose fermenting bacteria, other than the colon group which are encountered in water (10–11); (3) under-productivity in the case of

media using inhibitants, and (4) inherent lack of mathematical precision in the estimation of numbers of the colon-aerogenes organisms when grown in liquid media.

In testing water bacteriologically for evidence of fecal pollution, it is highly important to know the number of fecal organisms present, in order to estimate the extent of the pollution. Obviously, a strictly fecal type of organism should be used for such a test, if possible. It is known that the Bact. aerogenes type, when present in the human intestinal tract, is largely confined to the upper bowel, but at times reaches the lower intestine, usually as a result of accelerated peristalsis, that is, a diarrheal state, and thus appears irregularly in the feces along with the Bact. coli type more constantly present. The earlier conception, therefore, was that the aerogenes and the colon types of organisms had about the same significance as criteria of fecal polution.

With respect to the relative numbers of Bact. coli and Bact. aerogenes found in feces by the earlier methods, Levine (12) has summarized the results of 13 investigators and computed the average incidence of Bact. aerogenes in human feces to be 5.9 percent of the colon-aerogenes organisms present, in terms of the methyl red-Voges Proskauer reaction. In the individual reports, the percentages varied from 0 to 23.0 percent. Similarly, in animal feces the average finding of 11 investigators was 7.4 percent Bact. aerogenes, and the individual results ranged from 0 to 40 per cent.

In the earlier work, however, the relative numbers of colon and aerogenes types in feces were commonly estimated after growth of the mixed organisms in liquid enrichment media. Therefore, the relative numbers found do not necessarily reflect their numbers in the original sample. Ruchhoft et al. (1931) (13–14) have shown clearly what changes can be expected to take place in the initial colon-aerogenes relationships during growth of the two organisms in fermentative media.

They have shown that when equivalent numbers of Bact. coli and Bact. aerogenes are present in a water sample, it is impossible to predict which strain will outgrow the other in the enrichment broth. Ascendancy is functioned by the vigor of the strains and their sensitivity to metabolic products. Thus, studies of incidence, based on differentiation after the two organisms together have grown in a liquid fermentative medium, are apt to show a changed relationship from that which actually existed in the original sample.

The development of direct plating media greatly simplifies the quantitative estimation of colon-aerogenes organisms, and for the first time permits determination of their relative numbers in the original specimens. Our experience has been confined to one such medium, known as "ferrocyanide citrate agar" (15–17). In our hands this medium has proved to be a satisfactory means of counting and at the same time differentiating the colon and aerogenes types in feces, soils, water and sewage. In feces, it gives a distinctive picture of the colon-aerogenes relationships which, we believe, is of definite value in detecting fecal pollution in unknown samples.

In an examination of 521 specimens of human and animal feces by this "pour plate" method (18), the ratio of Bact. coli to Bact. aerogenes in the aggregate was found to be more than 100 to 1. In these specimens no instance was found of feces containing the Bact. aerogenes type alone. There were very occasional specimens of diarrheal origin, in which the Bact. aerogenes type predominated, but not to the exclusion of the Bact. coli type. On a percentage basis, the incidence of the Bact. aerogenes type in 150 specimens of human feces was found to be 1 percent of the colon-aerogenes organisms. In 371 specimens of animal feces, it was 0.7 percent. In the human and animal feces combined, it was 0.71 percent.

Keeping before us this later and more definite picture of the numerical relation between the Bact. coli and Bact. aerogenes types in feces itself, let us note the trend of opinion on the relative sanitary significance of Bact. coli. and Bact. aerogenes as criteria of fecal pollution.

Poe (1931) (19), in reviewing the work then available on the distribution of colon-aerogenes organisms in nature, expressed the view that if Bact. coli is predominantly of fecal origin and Bact. aerogenes primarily of non-fecal origin, the inclusion of both in a group test is inconsistent. In his opinion, the presence of the aerogenes group alone in a water supply is not sufficient evidence of pollution to justify condemnation of the supply. Mallman (1929) (20) points out that organisms having no relation to feces, belonging to the Aerobacter group, are sometimes confirmed in the Standard Method procedure as members of the colon-aerogenes group, and may thus cause the condemnation of safe water supplies. Ritter (1929) (21) advises caution in interpreting colon-aerogenes "group tests" of ground waters, and points out that Bact. coli disappears in stored sewage, while Bact. aerogenes survives for a longer time. She suggests that

the presence of Bact. aerogenes alone, in natural waters, may indicate contamination too remote to be of real sanitary significance.

Taylor and Martin (1929) (22) agree that a colon-aerogenes group test will condemn most tropical water supplies. They class organisms not utililizing citrate (Koser) as fecal, and regard them as more useful in judging fecal pollution. Lewis and Pittman (1928) (23) prefer the ferric ammonium citrate test to the methyl red-Voges Proskauer reaction, because the former correlates more closely with the sanitary quality of water. Rider (1928) (24), after correlating Bact. coli and Bact. aerogenes in water samples, with the sanitary survey, concludes that Bact. coli is directly indicative of fecal pollution and that Bact. aerogenes is associated primarily with surface contamination. Spray (1928) (25) states that of the 34 species comprising the colon-aerogenes group, the most confusing from the standpoint of interpretation in water analysis, are those of the Bact. aerogenes type.

The authors, in previous studies (18), have advanced the view that Bact. aerogenes as a natural inhabitant of soil and vegetation may find its way with the food into the intestinal tract of animals, including man, particularly with foods eaten raw, and hence may appear irregularly in the feces. This conception has impressed us as a more rational explanation of their occasional presence in feces, than the assumption that they are of direct intestinal origin.

In support of this view is the great predominance of Bact. aerogenes over Bact. coli in nature (20 to 1 in soils, grains and vegetation), as opposed to the predominance of Bact. coli over aerogenes in feces proper (100 to 1) (18).

The authors recently made a study (26) of the persistence of Bact. aerogenes and Bact. coli in decayed stumps exposed to the weather, and found that in the plantings of fecal material proper, Bact. aerogenes did not either outlast or overgrow Bact. coli. In the plantings from pure cultures, however, a strain of Bact. aerogenes, originally isolated from Lake Michigan water, occasionally overgrew the accompanying Bact. coli and became the predominating organism.

There is clearly a growing sentiment that Bact. aerogenes is not of much significance as a test for dangerous fecal pollution. If this view is correct, the conclusion follows that the colon-aerogenes group test of the Standard Methods of water analysis is not a specific indication of fecal pollution. The earlier references to "B. coli" in the literature mean, of course, the colon-aerogenes group, rather than Bact. coli proper, because the two types were not separated by the technic used.

As to the reliability of the differentiation of Bact. coli and Bact. aerogenes in ferrocyanide citrate agar, a high degree of correlation with the recognized differential tests has been obtained (18). The confirmatory tests used by us were, the methyl red-Voges Proskauer reaction, Koser's sodium citrate method, the ferric ammonium citrate agar method of Lewis and Pittman, and the eosin methylene blue colony differentiation of the Standard Methods of water analysis. The high degree of correlation found is interesting, but not essential to establish the value of a medium as a distinctive fecal test. None of these differential tests gave identical results and, of course, the same findings in different differential media are not to be expected, since the reactions depend on different biological properties of the organisms.

Even slightly different strains of an organism may be registered differently in different media. Hence a varying group of intermediate or variant strains between the two types of organisms classified as Bact. coli and Bact. aerogenes, may be expected, depending on the biological method used. The application of any fecal criterion implies a constant and unchanging method of differentiation in both the known fecal material and the unknown sample. Therefore, a given differential test, to be of greatest value as a fecal criterion, should be applied by itself to known feces, and to waters of unknown origin, without reference to confirmation by other tests depending on different biological reactions. To compromise one such test with another tends to confuse the relations and detracts from the value of the test. Just what differential method may eventually prove best as a test for fecal matter, remains to be seen, but it is our belief that each should be applied separately to both the known and unknown specimens alike, if they are to achieve their full measure of usefulness.

Ferrocyanide citrate agar in our hands gives four general types of colonies in pour plates made directly from the sample. One is the distinctive Bact. coli type; the second is the distinctive Bact. aerogenes type; a third is a group of intermediate or variant strains having some of the cultural characteristics of both. Members of this third or intermediate group are rarely, if ever, encountered in feces proper. They are found in sewage, though usually in relatively small numbers, but are most commonly found in soils, water and vegetation. The fourth group of colonies in this medium consists of miscellaneous organisms other than the colon-aerogenes types and is referred to as "extraneous growth."

In a large series of fecal specimens, this medium yielded the Bact. coli type of organism as the only one numerically worthy of consideration. In fact, the colon type of colony alone was characteristic of feces in this particular menstruum. The individual types of Bact. coli and Bact. aerogenes organisms subsequently cultivated in many successive generations were found to retain their definite colony characteristics in this medium for a long time—at least six months, as experimentally determined. Therefore, we believe that in applying the procedure logically as a fecal test, we should look for the Bact. coli type of colony alone, without reference to the aerogenes and intermediate types present in this medium, and irrespective of the possible variable reactions of some of these colonies, when grown in other kinds of media.

The authors, in a recent study of well waters in the Chicago district, found that the Bact. coli type of colony in ferrocyanide citrate agar, correlated much more closely and consistently with the findings of the sanitary survey (27) than did the Bact. aerogenes type of colony. The latter was associated with miscellaneous bacterial contamination, while the former had a more definite relation to fecal pollution.

CONCLUSIONS

1. There is a growing sentiment that the Bact. aerogenes types of organisms in water, apart from Bact. coli, have little significance as criteria of fecal pollution.

2. The fecal significance of organisms designated as Bact. coli and Bact. aerogenes depends on the differential method used to distinguish them.

3. In ferrocyanide citrate agar, the Bact. coli type of colony is the only one strictly characteristic of feces.

4. The Bact. aerogenes type and the intermediate types as determined by this medium, are inconsequential in feces.

5. The distinctive types of colon-aerogenes organisms in ferrocyanide citrate agar retain their definite colony characteristics for at least six months—as determined by subsequent plating of a large number of strains in the same medium.

6. It is the authors' conclusion that the Bact. coli type of colony in this medium is alone definitely indicative of fecal pollution; that the Bact. aerogenes type and the intermediate types of colonies are of no special fecal significance, and that the latter types should, therefore, be excluded from consideration as evidence of dangerous fecal pollution.

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DISCUSSION

F. E. HALE:[3] The question of the source of B. aerogenes is similar to the old question, Which is first, the chicken or the egg? When a child is born B. coli is absent from the colon contents but rapidly makes its appearance. Undoubtedly the source must be its food or conveyance from adult individuals. It is common testimony of the literature that both B. coli and B. aerogenes are present in the fecal matter of animals and human beings even though B. aerogenes is in the minority. It is also general testimony of the literature that the B. aerogenes types are able to persist under adverse circumstances. The predominance of B. coli or B. aerogenes, therefore, depends upon the environment. It appears to me logical that its source in grains and virgin fields may be either fertilizer used in cultivated fields and birds or animals in virgin fields. Its presistence then accounts for its predominance over B. coli.

B. typhosus dies out rapidly in water supply and even more rapidly in sewage although it may persist in soil contaminated with waste liquids for a year or more. In sewage it dies within a few days. In water supply 99 percent are dead in about two weeks according to Houston of London and other writers. In two months it is practically entirely gone from water supply. It seems important, there-

[3] Director of Laboratories, Department of Water Supply, Gas and Electricity, New York, N. Y.

fore, to know whether B. coli or B. aerogenes represents recent contamination or more remote. It is possible that B. aerogenes may generally denote remote contamination and therefore be of less significance from a sanitary standpoint than B. coli. There is no certainty of this in the test. The above would seem to offer the only purpose in identifying B. aerogenes separately from B. coli. However, as both are readily destroyed by chlorination there seems to be no advantage in separation of the forms.

B. welchii is probably also of sewage origin although it may be found in practically all soil. However, as it is a spore-former it is not readily destroyed by chlorination. There is, therefore, decided advantage in separating B. welchii from the B. coli group in the ordinary test, since the spores may persist in the soil and its presence is much more likely to represent remote contamination. As has been thoroughly demonstrated this is readily accomplished by the use of brilliant-green in the bile media.

Similarly the rapidity of formation of gas in liquid media gives a decided indication as to whether contamination is recent or remote. We have emphasized this point for many years. Experiments conducted at Mt. Prospect Laboratory and published in 1910, indicated that fresh B. coli produced gas in bile media within 24-hours up to a week. After a week it took two days for sufficient gas to show. After two to three weeks three days were required. Thus slow gas formation indicates remote contamination and a lapse of time which would indicate 99 per cent removal of any B. typhosus which might be present. In recent years, our Secretary, Mr. Jordan, had also emphasized the greater importance to be ascribed to early formation of gas in liquid media. Similarly Standard Methods for many years has not required confirmation of B. coli if gas occurred in lactose broth within 24-hours. This phase of the subject appears to me to be of far more importance than the separation of B. coli and B. aerogenes. In the next place there is no means of separating human from animal contamination and the greater part of that found in water supply is from animal sources. As animals do not have typhoid, only that B. coli coming from human sources may indicate danger of typhoid fever. In addition only a small percentage of human beings are typhoid carriers. We should avoid placing, therefore, too much emphasis on the presence of small amounts of the B. coli group, particularly when it is attenuated and probably indicates remote contamination.

There undoubtedly are particular instances where separation of B. coli from B. aerogenes may be useful. For example, in a swimming pool Levine found B. aerogenes and traced its source to the shoes of spectators alongside the pool. This lessened the significance of its presence in the pool. Similarly with open air pools where sand and dirt may blow into the pool it would be expected that B. aerogenes would be more commonly found and denote less significance. In fact, with open air pools we have frequently found gas production delayed to two or three days.

There is also another phase of the subject. B. aerogenes is just as definite an indicator of soil-wash as is B. coli. Many water supplies are of surface character and obtained from watersheds. B. typhosus deposits on watersheds have been the cause of the greater portion of serious typhoid epidemics due to water supply. As previously stated the B. coli group of itself is of little significance except as indicating the possibility of typhoid, and the fact of soil-wash as caused by heavy storms is what we are really interested in.

Since receiving Dr. Tonney's complete paper I have been struck by one sentence "There were very occasional specimens of *diarrheal* origin, in which *Bact. aerogenes* types predominated, but not to the exclusion of Bact. coli type." In the printed annual report for 1930 of the Division of Laboratories and Research of the New York State Department of Health, on page 31, is described an out-break of food-poisoning apparently due to Cream Puffs. From the filling of all puffs examined but one and also from five of seven specimens of feces from patients who had been acutely ill B. lactis aerogenes was isolated and filtrates of a medium containing ingredients corresponding to those in the cream-puff filling in which this culture had been grown for twenty-four hours were toxic for rabbits and chickens. Within one to three hours after feeding these animals developed diarrhea. Other bacteria isolated failed to show evidence of toxigenic properties. Chemical poison was not found. None of the species of organisms associated with enteric disease were isolated from puffs or feces. The additional statement is made that since B. lactis aerogenes has not been definitely associated with out-breaks of food-poisoning in the past, a detailed study of this strain seemed warranted. In view of the above there is a question as to whether B. aerogenes may not have a special significance at times.

Dr. Tonney describes four groups of colonies which grow upon ferrocyanide citrate agar: Distinctive Bact. coli type, distinctive

Bact. aerogenes type, variable intermediate strains, and extraneous growths not of the colon-aerogenes types, and recommends that only distinctive Bact. coli type be considered as alone definitely indicative of fecal pollution. This would certainly be a radical departure and a questionable policy.

We are of the opinion that the use of solid media in confirmation of the B. coli group from gas-formers in lactose broth has resulted in much of the confusion which has arisen, due to the attempt to pick typical B. coli in this confirmation and yet typical B. coli and typical B. aerogenes grow very distinctively on eosin-methylene-blue agar. We are uncertain whether such a complex medium as ferrocyanide citrate agar *in general use* would find any better fate.

THOS. R. LATHROP:[4] There is perhaps no one test in the bacteriological laboratory which is so unsatisfactory as the analysis of water. Other diagnostic methods have been developed to the point that in most cases very definite statements can be given regarding the pressence or the absence of the organism sought. On the other hand, results of examination of water from miscellaneous sources not only place the bacteriologist in a dilemma, but it seems that the lay mind must often discredit the work of an analyst who of necessity must give reports which, to say the least, are frequently very indefinite.

The purpose of the bacterial analysis of water is to detect fecal pollution. We all know that the present "Standard Methods" have fallen far short of having provided a means of attaining that purpose in a reasonable length of time, due to the inclusion of many organisms in the coli-aerogenes group which are of non-fecal origin and due to the lack of media of sufficient specificity to differentiate between the fecal and non-fecal types.

While this has presented great difficulties in the laboratory where miscellaneous waters are analyzed, due to the uncertainity in the mind of the analyst as to the exact meaning of the analysis, there has been a less marked difficulty at the water purification plant. Here it has been found possible with ordinary careful operation to produce a plant effluent free from the coli-aerogenes group. It is true, however, that the lack of specificity of lactose fermentation tests have been the cause of a great amount of work in making confirmations of the many lactose fermenters encountered in various steps of the purification process.

[4] Assistant Engineer, State Department of Health, Columbus, Ohio.

A limited amount of work in the laboratory of the Ohio State Health Department, employing the ferrocyanide-citrate agar developed by Noble, has demonstrated that a differentiation of coli and aerogenes is reasonably simple. The time required is short. The work involved is much less than that required for completed tests by the present standard methods. The enumeration of the organisms is much more accurate than under the present admittedly cumbersome and inaccurate method of enumeration.

The ratios of coli to aerogenes which are present in fecal and non-fecal material as reported by the authors and others would seem to make it inconsistent to include the aerogenes and coli in a group test. There is no question that the presence of the B. coli group in a water is indicative of sewage pollution. A very serious question has been raised by the authors as to the significance of B. aerogenes as criteria of sewage polllution.

The differentiation tests employing ferrocyanide-citrate agar by which these two groups can be separated seem worthy of study by other laboratories. It is possible that in this medium we have the long sought for method for examination of water.

The authors have found that B. coli are the only organisms of numerical importance in fecal specimens. Many investigators have found few or no aerogenes in feces. Their conclusion, therefore, seems well taken that in the examination of waters for fecal pollution consideration should be given only to the B. coli type of organisms.

FRED O. TONNEY:[1] In connection with Dr. Hale's somewhat theoretical discussion I want to bring out one or two essentials.

The direct plating method referred to serves primarily for the group count of colon aerogenes organisms. The fact that at the same time it differentiates the two main types of the group does not detract from the value of the group count or render it in any way less dependable for those who desire it. In fact, a direct colony count is inherently more likely to be accurate as a quantitative procedure than an estimate of numbers in liquid media.

That Bact. coli is the predominating organism of feces is universally accepted, and that it is generally more resistant than Bact. typhosus and related pathogens, to environmental factors at large, is also well known. Therefore, the exclusion of an organism of doubtful fecal origin, such as Bact. aerogenes, from consideration as a fecal criterion, does not jeopardize the detection of fecal pollution. Bact. coli,

always present in fecal material, may be relied upon to do this. The exclusion of Bact. aerogenes, however, does eliminate strains of the latter organism widely present in nature in great excess of their normal fecal ratio to Bact. coli, and to this extent renders the test more specific of direct fecal pollution.

The value of Bact. aerogenes as an indicator of soil wash is accepted by us. In that respect it seems to run about parallel to the total bacterial content of water. Our experience leads us to believe, however, that the separate Bact. coli count adequately indicates soil wash of a fecal nature.

The injection of cream puff intoxications at this point does not appear germane to the issue. Cream puffs are subject to spoilage by growth of numerous and varied organisms. Public health laboratories frequently encounter these outbreaks, the specific bacterial origin of which is obscure and extremely difficult to determine. The inference that an organism such as Bact. aerogenes, widely present in all foods eaten raw, can be an intestinal pathogen of any serious consequence, does not seem reasonable; nor does such evidence point to it as of primary fecal origin.

We are in agreement that the Bact. coli and Bact. aerogenes types of organism are of about equal value as a test for effective chlorination in plant control. The "chlorine death points" of these two organisms as previously determined by us were not essentially different and were in general greater than the common vegetative pathogens of the typhoid-dysentery group. We feel that the separate differential counts probably have a greater field of usefulness in judging the quality of raw waters than of treated waters.

Mr. Lathrop's discussion interests us as representing the viewpoint of one who has actually used the direct plating method in practice. With use of the procedure, many preconceptions and misunderstandings of the technique and its findings disappear, and the method is usually found to be easy of application and less complex than most similar laboratory procedures now in vogue. It is by just such practical applications of the method to the problems of water analysis, that its ultimate usefulness and the significance of the additional data it affords, can be evaluated.

CONFORMITY OF PUBLIC WATER SUPPLIES TO THE UNITED STATES TREASURY DEPARTMENT STANDARDS

By R. E. Tarbett[1]

In discussing the subject of conformity of public water supplies to the Treasury Department Standards for drinking and culinary water supplied by common carriers in interstate commerce it would seem desirable to consider briefly (1) the development of the standards (2) the standards themselves and (3) their application by the State health departments and the Public Health Service.

The certification of drinking and culinary water supplied by common carriers is based on what is commonly called the Quarantine Law of 1893. This law authorized the control activities necessary for the prevention of the entrance and spread of communicable diseases into the United States from foreign countries and from one State into another. In so far as interstate spread of disease is concerned the law directs that control be exercised by State and municipal agencies in coöperation with the Federal public health service when such agencies will undertake this task. That section of the law applying particularly to the prevention of interstate spread of disease reads as follows:

"Sec. 3. That the Supervising Surgeon General of the Marine Hospital Service shall, immediately after this act takes effect, examine the quarantine regulations of all State and municipal boards of health, and shall, under the direction of the Secretary of the Treasury, coöperate with and aid State and municipal boards of health in the execution and enforcement of the rules and regulations of such boards and in the execution and enforcement of the rules and regulations made by the Secretary of the Treasury to prevent the introduction of contagious or infectious diseases into the United States from foreign countries, and into one State or Territory or the District of Columbia from another State or Territory or the District of Columbia. . . . . The Secretary of the Treasury shall, if in his judgment it is necessary and proper, make such additional rules and regulations as are necessary to prevent the introduction of such diseases into the United States from foreign countries, or into one State or

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Territory or the District of Columbia from another State or Territory or the District of Columbia, and when such rules and regulations have been made they shall be promulgated by the Secretary of the Treasury, and enforced by the sanitary authorities of the States and municipalities, where the State and municipal health authorities will undertake to execute and enforce them."

Under this authority the Secretary of the Treasury has from time to time issued regulations known as the Interstate Quarantine Regulations.

In view of the inadequate control, which was being exercised by the health authorities over water supplies used by common carriers the Secretary of the Treasury promulgated regulations in 1912 requiring that all common carriers file with the Surgeon General of the Public Health Service a certificate of purity of water used by them. This certificate was to be obtained from State or local health officer. Needless to say practically all water supplies regardless of quality were given a clean bill of health. Since such a condition did not actually exist it was realized that some standard was necessary by which the quality of a water supply might be judged. A commission of competent bacteriologists and chemists was appointed to formulate standards and this commission made its recommendations in 1914, the standards recommended being promulgated by the Secretary of the Treasury under date of October 21, 1914.

This Commission realized that the quality or safety of a water supply could not be determined by an occasional bacteriological examination. However, at that time the machinery for proper inspection of supplies was entirely absent in a majority of the States, only 14 States having engineers connected with their State health departments. It was felt that carrier companies could, where occasion required, disinfect the supply necessary for their use. Consequently only bacteriological standards were recommended. These standards were rather severe and very few public supplies could meet them. The standards were not intended for adoption as standards for public water supplies in general and the Commission so stated. However, as might be expected, they were quite generally accepted as such.

In 1919, the Public Health Service, having carried on a considerable amount of work and study in connection with carrier drinking water supplies, proposed a coöperative procedure for certification. The State health officers agreed to coöperate and the present procedure was inaugurated. Realizing that many of the States were without

engineering personnel with which to make surveys of supplies the Service assigned engineers to these States with two objects in view. One to inaugurate certification procedure; and two, to foster the establishment of engineering divisions in the State health departments. The effect of this latter development is seen when we consider that in 1925, when the present standards went into effect, 42 States had established engineering divisions in their departments of health.

With the establishment of the new procedure which was outlined in the revised regulations of 1921, more comprehensive standards were necessary. In 1922, another committee was appointed to revise the existing standards. The machinery was at that time available for carrying on the surveys and laboratory examinations necessary for properly judging the quality of water used for drinking purposes. Consequently the requirements included those relative to the source and protection of water supplies, as indicated by a careful sanitary survey and other requirements relative to bacterial content, as shown by standard tests. While the bacteriological standards were stated in definite terms the committee recommended that in actual practice they be applied with some latitude. Supplies which on rigid inspection were found to be satisfactory in all other respects but failed to meet the bacteriological standard might be accepted in the discretion of the certifying authority.

These standards as recommended were adopted in 1925 and while not intended to apply to water supplies generally have been so accepted.

In considering compliance with the Standards it is quite common to consider only the bacteriological standards. The fact that the determination of whether or not a supply conforms to the Standards is dependent more upon "Source and Protection" rather than upon the results of the laboratory examination is often overlooked.

It may not be out of place at this point to state the Standards with which as a whole many of you may not be acquainted:

STANDARDS

1. As to source and protection

(1) The water shall be:—

(a) Obtained from a source free from pollution; or

(b) Obtained from a source adequately protected by natural agencies from the effects of pollution; or

(c) Adequately protected by artificial treatment

(2) The water supply system, including reservoirs, pipe lines, wells, pumping equipment, purification works, distributing reservoirs, mains and service pipes, shall be free from sanitary defects.

II. As to bacteriological quality

(1) Of all standard (10 cc.) portions examined in accordance with the procedure specified below, not more than 10 percent shall show the presence of organisms of the B. coli group.

(2) Occasionally three or more of the five equal (10 cc.) portions constituting a single standard sample may show the presence of B. coli. This shall not be allowable if it occurs in more than:

(a) Five percent of the standard samples when 20 or more samples have been examined;

(b) One standard sample when less than 20 samples have been examined.

III. As to physical and chemical characteristics

The water should be clear, colorless, odorless, and pleasant to the taste, and should not contain an excessive amount of soluble mineral substances nor of any chemicals employed in treatment.

The committee outlined in its report procedures to be followed and listed at length the undesirable features that should be looked for.

Under the present procedure of certification inspections are made by representatives of the State departments of health and recommendations are made by the State health officers to the Surgeon General. In making these recommendations the tendency for the past few years has been toward more complete compliance with all sections of the Standards. This has been a natural development on the part of the State health organizations and not due to pressure brought by the Public Health Service. This has been particularly true in connection with the section of the Standards having to do with "Source and Protection" and is shown by the increase in provisional certificates recommended. There may be some differences of opinion in the various States as to the potential danger where certain undesirable sanitary defects exist. This may bring about a favorable recommendation in one State and provisional in another for a similar supply. If this difference of opinion were great it might have a material effect on the uniformity of certification of supplies by States. However, with the possible exception of the question of adequate protection of distributing reservoirs located within the community, such is not the case.

CONFORMITY WITH STANDARDS

With these remarks relative to standards and certification behind us, let us consider the question of conformity of public water supplies with these Standards. Such observations as can be made must naturally be based on water supplies used by interstate carriers. The number of these supplies is only a small percentage of the total supplies in the United States. However, it does represent a sufficient number in each State to allow some conclusions to be drawn as to the whole.

The number of public supplies listed by the carriers during the past seven years has been fairly constant, averaging about 1950. During this same period there has been a reduction of 30 percent in the number of non-public supplies used.

The public supplies listed by States vary from approximately 200 in Texas to 3 in Rhode Island.

The records kept do not permit an analysis concerning public water supplies only. However, a study of the records for 1931 has been made from which it is possible to arrive at some conclusions as to conformity to the Standards as well as reason for non-conformity. Of the 1944 public supplies listed for 1931 reports were received on 1836. Of this number 79 percent were favorably certified, 18.5 percent provisionally certified and 2.5 percent prohibited for use.

It may be of interest to you to know that of these 1944 supplies, 31.1 percent received treatment other than plain chlorination and 20.4 percent employed chlorination only, a total of 51.5 percent receiving some form of treatment.

For the most part, the supplies favorably certified have met the conditions required under the Standards. In some instances where compliance with the bacteriological Standards has been lacking consideration of compliance with the Standards as a whole warranted approval. Continued failure, however, to meet the bacteriological standards will bring about provisional certification.

Since there is some added incentive for water works officials in charge of supplies used by carriers to make every effort to meet the requirements, it might be expected that a larger percentage of these supplies would conform to the Standards than would otherwise be the case. As applied to public water supplies as a whole it is not believed, however, that the percentage of 79 needs be discounted to any great extent. The existence of the Standards is generally known; there is a certain amount of pride in conforming to these standards; and the

State department of health, the same authority that determines the certification status, has certain jurisdiction over all public supplies. That 75 percent of the public water supplies within the United States conform to the essential requirements of the Standard does not appear an unwarranted assumption.

Reasons for prohibiting the use of a supply need little consideration. Each year between 2 and 3 percent of the supplies listed are prohibited for use. In the case of non-public supplies permanent elimination generally follows. With the public supplies corrections are made and the supply is again placed on the accepted list.

A supply prohibited for use generally fails to comply with both the source and protection standards and with the bacteriological requirements, although the latter may not always be the case.

In 1931, 43 percent of the public supplies prohibited, based on the number of laboratory examinations made, met the bacteriological standards. Thirty percent of these were prohibited because of improper protection of ground water supplies, such as location, construction and lack of or inadequate treatment.

PROVISIONAL GROUP

Perhaps the most interesting group to consider is the provisional group since it gives an indication of the trend of official opinion as to what constitutes an undesirable defect warranting a provisional recommendation. As stated before there are some slight differences of opinion among the officials of the different States which tend to establish somewhat higher standards in some States than in others. Recommendations for provisional certification in 1931 indicate, however, that these differences of opinion are becoming less and that rather rigid compliance with the Standards is necessary for favorable recommendation. This is evident in the number of supplies provisionally certified. The average of supplies provisionally certified for the year 1925 to 1929 inclusive was 9.5 percent of the total number of supplies reported upon, whereas, for 1930 and 1931 this percentage became 19.5. The percentage of public supplies thus certified in 1931 was 18.8.

A study of the reports on public supplies provisionally certified shows that a number of defects other than failure to meet the bacteriological standards are the cause for such certification. These include non-approved cross connections with other supplies, inadequate treatment of water, inadequate protection of ground water supplies, inadequate protection of distributing reservoirs or tanks,

poor operating control, lack of proper watershed protection, failure to submit samples for examination and reports of operation to the State departments of health, as well as others.

In only 22 percent of the cases did the supply fail to meet the bacteriological standards and in only 5.7 percent was this the only cause for the provisional recommendation. If we consider the provisionally certified supplies in connection with those favorably certified we find that approximately 90 percent of the public supplies reported upon met the bacteriological standards.

The reports on supplies recommended for provisional certification show the presence of a few principal undesirable features which alone or in combination with others were the cause for such recommendation. These are of sufficient importance to bring to your attention. In 22 percent of the supplies the water failed to meet the bacteriological standards as judged by the limited number of examinations made. In 34.2 percent unapproved and uncontrolled cross connections existed. In 22.5 percent necessary treatment was lacking or inadequate. In 28.2 percent ground water supplies were not adequately protected insofar as location, construction and equipment were concerned. In 21.7 percent distributing reservoirs and tanks were not protected by covering or chlorination or were liable to surface or sub-surface pollution. In 18.2 percent inefficient operating control existed.

Under that section of the Standards having to do with Source and Protection any one of the above was considered of sufficient importance to warrant the provisional recommendation.

Summarizing briefly we find that in 1931, 1944 public water supplies in the United States were used as sources of drinking and culinary water by interstate carriers. Seventy-nine percent of the supplies reported on were recommended for favorable certification, 18.5 percent for provisional certification and 2.5 percent prohibited for use.

It can be assumed to all intent and purposes practically all supplies favorably certified meet the bacteriological standards as did 78 percent of those provisionally certified. If the supplies listed for certification may be considered as a cross section of all public supplies it would be expected that at least 75 percent of all public supplies would meet the Treasury Standards as a whole and that approximately 90 percent would meet the bacteriological standards.

More rigid adherence to the Standards reacts favorably in not only placing the water supplies involved, but all others as well, on a higher sanitary plane with corresponding greater safety of the water produced.

NUMERICAL RATING OF PUBLIC WATER SUPPLIES

By E. Sherman Chase[1]

The alchemist sought the philosopher's stone whereby to turn lead into gold. The Spanish soldier of fortune explored the New World for the fountain of perpetual youth. So, today, does the sanitary engineer seek a single standard by which to judge the quality of water supplies. Perhaps the modern quest is as little likely of success as those other endeavors of the past.

In spite of the difficulties inherent in such a task, numerous attempts have been made from time to time to set up standards for water supplies. The best known of such standards is that of the Treasury Department for drinking water used on inter-state carriers. Many years ago, before the advent of bacteriological methods, certain limits with respect to the organic content of waters used for potable purposes were employed for classifying water supply quality. The development of bacteriological methods brought to the engineer a most useful tool for determining the safety of water for drinking purposes. Today, water supplies are judged very largely upon their freedom from organisms of the B. coli type, or to use the more modern nomenclature, the coli-aerogenes group. For example, the Treasury Department standard is rather generally thought of as a standard of B. coli, although, as a matter of fact, other conditions must be met as regards chemical constituents and source. It is not difficult to understand this attitude. It is so easy to say, "A safe water is one which does not contain B. coli in excess of 1 per 100 cc."

Irrespective of differences of opinion as to whether or not such a single analytical standard is too severe, a single criterion of quality based on analytical methods may fail to give even a reasonably true picture of the relative public health hazards of different water supplies. For instance, two supplies may show equally satisfactory B. coli counts, but in one case there are unprotected cross connections with polluted industrial supplies and in the other case, none. Obviously, the two supplies offer different health hazards. Another example might be given. Two supplies are from practically similar sources, each provided with modern water treatment devices, but

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in one municipality the water works employee's tenure of office is dependent upon political whims and in the other, upon the employee's own ability and skill. Here again there is a marked difference in the potential danger to public health.

The standard of the Treasury Department involves requirements much more inclusive than the one relating to B. coli. The standard also covers the sources and protection of supplies and their physical and chemical characteristics. No attempt, however, is made in this standard to weigh the relative significance, from the standpoint of public health, of the many factors involved. While it may have been somewhat presumptuous, the author of this paper, for a rather longer number of years than he cares to disclose, has been working upon and applying a systematic scheme for the numerical rating or scoring of public water supplies from the standpoint of public health.

In 1927, the New England Water Works Association adopted as a tentative working system a scoring method recommended by a committee consisting of A. D. Weston, now Chief Engineer of the Massachusetts Department of Public Health, J. F. Jackson, formerly chief engineer of the Connecticut Department of Health and the author. At the Montreal meeting of the American Public Health Association in the fall of 1931, a committee was appointed to work with similar committees of the State Conference of Sanitary Engineers and of the American Water Works Association to study the feasibility of working out a score system relating to the sanitary quality of water supplies which would be fairly generally applicable to supplies throughout the United States with a reasonable degree of accuracy.

Except for a few conferences between the chairmen representing the three organizations, little has been accomplished to date, other than the establishment of a closer understanding of the several points of view regarding the subject under consideration. The object of this paper, therefore, is to bring the subject before the American Water Works Association in order that the Association as a whole may have a better conception of the problems involved and of the merits and demerits of any numerical rating of the sanitary quality of water supplies.

REASONS FOR A GENERALLY APPLICABLE SCORE SYSTEM

The primary object of any score system or standard of quality is to help bring about the improvement of inferior water supplies and

the maintenance of the quality of good supplies. A score system can be made to be an effective tool in the hands of the health department seeking to bring about water supply improvements. Judiciously used, comparative scores may be employed to stimulate competition in water supply betterments between rival municipalities. Furthermore, the keeping of scores is a valuable aid in the administrative work of the health department by emphasizing the supplies needing the greatest improvement and consequently the communities upon which the most pressure should be brought for such improvements.

For the active, intelligent, up-to-date water works superintendent, a score system affords a means of casting up the balance sheet of sanitary assets and liabilities relating to his water supply. Properly and tactfully used, such a balance sheet may be used by the superintendent to secure funds for needed improvements when the score is low. When the score is high, it can be referred to as a matter of civic pride and as an argument for the continuation of adequate appropriations.

In the case of the consulting engineer, the score system provides a method by which the relative merits of different sources of supply may be compared upon a definite and systematic basis. The mere cataloging of the numerous factors affecting the safety of the sources of supply under consideration may lead to a more comprehensive understanding of their relative desirability. Consequently, the score system becomes a valuable aid to judgment for the engineer.

OBJECTIONS TO A SCORE SYSTEM

There are several more or less valid objections to the adoption of any numerical rating system for water supplies. The principal ones are discussed briefly.

Probably the most valid objection to any numerical system of scoring is that the values assigned the many factors are purely arbitrary and have little scientific basis. This criticism must be acknowledged as a just one. On the other hand, it should be recognized that the values selected must be based upon experiences with a great many water supplies. It must also be appreciated that the rating of a water supply is not a precise determination any more than the pacing of a distance is a precise measurement; both, however, are approximations of the truth and distinctly better than unaided judgment.

Another valid objection is that the application of any score system

is subject to the individual viewpoint of the person doing the scoring. With a properly devised system, this objection may be largely, if not entirely, obviated. It is an objection, however, which must be borne in mind.

A third and rather serious criticism is that the data required for the preparation of a numerical rating are often very limited, both in scope and in accuracy. This lack of data does constitute a serious handicap in the preparation of reasonably accurate scores. This objection, however, seems also as an excellent argument for a score system. The data called for should be known. Facts are essential and unless the facts are known, it would be folly to hazard a guess or to express an opinion as to the safety of a public water supply. If a score system serves to bring out the facts, that function alone makes such a system well worth while.

Still another objection, and, to the author, an invalid one, is that the working out of a score is a tedious job involving the expenditure of considerable time and effort. When consideration is given to the time required in making sanitary surveys and in making innumerable analyses, it seems rather ridiculous to object to the expenditure of an hour or so in rounding out and summing up a study of a water supply.

Score systems have also been objected to on the grounds that the results may cause annoyance to local officials, thereby bringing about the exertion of political pressure for modification when such scores are prepared by state health departments. An objection of this character is based upon the proverbial policy of the ostrich in the face of danger, and deserves but little consideration.

To the author, the preceding objections do not appear insurmountable. Similar objections apply to rating schedules dealing with other subjects. Score systems for dairies, pasteurizing plants and municipal health activities have been applied with success. Probably the most effective rating system is that of the Fire Underwriters. Every objection raised against the scoring of sanitary hazards applies equally well to the scoring of fire risks. Yet the Underwriter's schedule is eminently successful in bringing about needed betterments in fire protection service.

ADVANTAGES OF A SCORE SYSTEM

Some of the advantages of a score system have already been dwelt upon in considering the reasons for such a system. There are a number of others deserving of mention.

One distinct advantage is that a score system shows up not only active but potential contamination as well. Bacterial analyses show past and recent occurrence of contamination but knowledge of the facts connected with the source, protection and distribution of a supply required for the purpose of scoring permits an appraisal of the possibilities of future contamination.

While it must be clearly understood that the values used in a score system are arbitrary and based upon judgment and experience, the final summation in terms of percent affords a means of conveying to the laymen definite concepts of relative sanitary quality.

Another way in which a score system is of value is that it involves a list of many factors which should be known if a true picture of the general safety of the supply is to be obtained. For investigators of public water supplies the requirement for the preparation of a score tends to bring about a more thorough investigation than might otherwise be obtained.

Although the quality of the water delivered as determined by analyses may be considered by some as the only real criterion of the safety of a supply, it must be appreciated that for analyses to be of the greatest value they must be frequent and regular. In the case of many public water supplies, analyses are few and far between. Although analytical results are given due weight in the score system, they constitute only part of the factors involved. Consequently, it is possible to secure a better understanding of the relative safety of a supply from the public health viewpoint by means of the score system than by means of a few scattering analyses.

NUMERICAL RATING SYSTEM PROPOSED FOR CONSIDERATION

The numerical rating system which the author submits for consideration is substantially the tentative system adopted by the New England Water Works Association with certain modifications indicated as desirable by further study and experience. For convenient reference the system in detail is appended.

The factors affecting the safety of a public water supply from the health standpoint may be divided into three major groups, first, pollutional hazards connected with the source; second, measures adopted to protect against sanitary hazards; third, the proof of the pudding, or the quality condition of the water as delivered to the consumer.

Under each of these major groups are listed sub-factors which

have bearing upon the continued safety of the supply. For those factors which constitute sanitary hazards, numerical penalties are assigned, roughly proportional to the degree of hazard involved. For those factors which protect or improve the supply, numerical credits are given, also roughly proportional to their relative value and efficiency. Credits and penalties are set up for the several quality conditions of the water as delivered, these credits and penalties being assigned values according to their relative significance.

In rating supplies, it is evident that surface supplies and ground water supplies must be treated differently. For this reason the score system has been so arranged as to deal with these two groups separately. Furthermore, it is obvious that the supplies from the Great Lakes and connecting rivers must constitute a special sub-group in the main group of surface supplies. In the case of the ground water supplies, these supplies are divided into two-sub-groups, namely; wells having tight casings less than 200 feet deep and wells having tight casings more than 200 feet deep. For convenience these groupings are tabulated as follows:

1. Surface supplies:
   a. Main group.
   b. Sub-group—Great Lakes, connecting rivers and St. Lawrence River.
2. Ground water supplies:
   a. Wells having tight casings less than 200 feet deep.
   b. Wells having tight casings more than 200 feet deep.

A detailed discussion of each of the various factors and of the numerical values assigned them would be too long for presentation. The author in other places has discussed these matters at some length and the following excerpts from a previous paper on the same subject serve to indicate some of the reasoning involved in the preparation of the system:

"In general the numerical values ascribed to the several factors under each of the major divisions have been selected with the view of securing final scores within a range of 0 to 100. In some cases it will be found that the score will fall outside the range, but it is believed that nearly all supplies will score greater than 0 and less than 100.

'Wherever there is resident population upon the watershed, there is a pollution hazard varying in general in proportion to the density of that population. The hazard resulting in the direct discharge of sewage into a water course is obviously much greater than the hazard from a population whose excretal wastes are discharged into cesspools or ordinary privies. It is true, however,

that serious epidemics of water-borne disease have resulted from the infection of supplies derived from sparsely inhabited water-sheds where no direct sewage discharge occurred. Generally speaking, however, the hazard of indirect pollution, socalled, is substantially less than that from direct pollution.

"The location of population upon drainage areas with respect to water supply intakes is a factor which affects to a very marked degree the relative pollution hazard. It has, therefore, been considered essential to divide watershed areas into zones of multiples of 50 miles upstream from the intakes. One reason for selecting 50 miles is that the U. S. Public Health Service in its studies of the pollution and natural purification of the Ohio River divided that watershed into 50-mile zones above the intakes of the Cincinnati and Louisville water supplies. The data of the Ohio River study have been of much value in arriving at the proper allowances to make for the proximity or remoteness of sources of pollution to water supply intakes. In the accompanying score system, the variation in weight given to population in the different zones approximates the "die-away" curve of B. Coli in the Ohio River under winter conditions. The majority of water supplies are derived from such small watersheds that zoning is unnecessary. While it is true that even on small watersheds the relative location of sources of pollution to the intake affects the hazard involved, it is impracticable to attempt any differentiation between remote or near sources of pollution in such cases."

"Industrial wastes have a variable effect upon the quality of water supplies according to the kind of wastes and the degree of dilution afforded. For example, a very small quantity of phenol wastes will have a marked effect upon the quality of chlorinated water, whereas relatively large volumes of other wastes will have small effect. It is very difficult to fix a detailed gradation of penalties for industrial wastes and the reasonable course to adopt appears to be the selection of a maximum penalty, leaving to the judgment of the individual scoring the supply the penalty to apply in any one case.

"It is practically impossible today to secure a water supply of pristine purity. Nearly all water supplies are subject to some source or mode of pollution. Natural barriers are imperfect and unreliable. To secure adequate protection artificial barriers must be established and carefully controlled.

"First in order of protective measures are those relating to the abolition or abatement of sources of pollution. Treatment of sewage and of industrial wastes is of value in the reduction of pollution in proportion to the degree of treatment afforded. These measures, however, are more logically considered as being means for reducing the intensity of the pollution than as means for protecting the water supply.

"Storage as a protective measure has long been recognized and in New England is the most frequently met with and relied upon protective measure. Most water-borne infectious bacteria die off in two weeks' time and their destruction in a month's time is almost certain. Storage, although it cannot be relied upon implicitly, due to the uncertainty with respect to the actual degree of reduction secured and the practical improbability of absolute control, is of considerable value Increasing draft or depleted reservoirs tend to reduce the value of storage. Furthermore, there is in some cases serious danger of con-

tamination being brought promptly to the intake as a result of short circuits or other currents which render the time of passage from point of pollution to intake relatively short. Another drawback to long storage lies in the possibility of the undue development of algae growths which render less efficient or more difficult subsequent purification processes.

"Methods for water purification by artificial means are obviously important considerations. Credits for such methods must, of course, be based upon judgment as to the relative reliability of the several measures. Where a final score would exceed 100, the total credit for protective measures should be limited to a definite figure in as much as measured may be adopted for which the situation does not call. Accordingly an excess of purification factors ordinarily deserves no credit. Under some cases where comparisons are being made of different sources of supply, full credit for such protective measures should be given even though the final score might exceed 100.

"Purification and other protective measures even though provided do not always function as planned and accordingly provision should be made for reducing the credits for such devices when they are inadequate or improperly operated and controlled.

"The final test of water supply is the quality of the product delivered. The consumer demands a clear water, low in color and without odor or taste. A soft but non-corrosive water is desirable and one free from iron and manganese. From the sanitary standpoint, physical and chemical qualities, although having an indirect bearing upon the health of the community, are relatively less important than the bacterial qualities. It is a well known fact that where the public water supply is distasteful the consumers frequently resort to other sources of supply of doubtful sanitary quality.

"The most important consideration with respect to the quality of the final product is its bacterial content and the score system proposed is arranged accordingly. Although the bacterial standard of the Treasury Department might be used, it does not seem wise to do so because this standard draws so sharp a difference between acceptable and unacceptable supplies that it is of no value for the preparation of relative ratings. The primary consideration is the presence or absence of pollutional organisms of the B. coli type, although some weight is given to total bacterial counts on agar at 37°C

"In the case of the Great Lakes, St. Lawrence River and connecting rivers, it is evident that on the large watersheds tributary to these bodies of water, the remote populations have no effect upon the quality of the supply. Limiting distances have been selected therefore based upon the probable extent to which pollution will travel in these bodies of water. In so far as protective measures and quality conditions are considered, the same credits and penalties are applicable as for the main group of surface supplies.

"As regards ground water supplies, the proximity of population and of pollution must be taken into consideration. It is evident that the materials through which wells pass are of considerable importance as is the depth of such materials, so that varying credits must be allowed according to the material penetrated. In effect, the passage of ground water through the soil is a purification process and the reliability and extent of the process will de-

pend both upon the depth of the material and its character. These points must be taken into consideration. It is also evident that credits and penalties for artificial purification methods and for quality conditions should be the same as for the other groups, although it has been argued that pollutional organisms found in ground water supplies are of greater significance than when similar organisms are found in the surface supplies. There are, however, inadequate data to substantiate this opinion.

In conclusion, it should again be stated that the score system is a yardstick whereby the relative safety of public water supplies may be appraised. It is an approximate weighting of many unlike factors, not a precise index based upon a single factor. It is a useful tool, but one which must be used with good judgment and thorough understanding of the principles involved. Its faults are evident, but all in all, if properly used, it should be a means for the formation of sound and correct opinions as to the sanitary quality of public water supplies. To be of the greatest value, it should represent as closely as possible the collective judgment of water supply chemists, bacteriologists and engineers from all parts of the country as to good water works practice. For this reason, the author recommends its careful consideration by the American Water Works Association.

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APPENDIX

OUTLINE OF TENTATIVE SCORE SYSTEM[2]

Surface supplies (main group)

I. *Pollution Hazards:*

Score for no hazard 60

Base Penalties for Hazards connected with Sources of Supply:

1. Population on watershed in 50 mile zones of intake:

   p_1 = Population in 1st zone per sq. mile of total drainage area.

   p_2 = Population in 2nd zone per sq. mile of total drainage area, etc.

   a. Contributing indirect pollution or where sewage is diverted from drainage area:

   $$-2\sqrt{p_1 + 0.7\,p_2 + 0.39\,p_3 \text{ etc.}}$$

   b. Contributing direct pollution (sewage):

   $$-8\sqrt{p_1 + 0.7\,p_2 + 0.39\,p_3, \text{ etc.}}$$

   c. Contributing direct pollution but with sewage treated divide penalty (b) by following factors according to degree of treatment:

      1. Sedimentation, 1.5
      2. Complete treatment without disinfection, 3.
      3. Complete treatment with disinfection, 4.

   In case of improper operation of sewage treatment plants use no divisor.

2. Highways and railroads (per linear mile per sq. mile of watershed)[3] ..... −1
3. Lumbering, labor camps, hunting, etc.:[3]
   a. Lumbering, labor camps, etc. ..... −2
   b. Hunting, picnicking, etc. ..... −1
4. Animal pollution:[3]
   a. Direct ..... −2
   b. Indirect ..... −1

[2] Modification of system adopted by N. E. W. W. A., September 13, 1927.

[3] Applied only when population is less than 100 per sq. mile.

5. Industrial wastes	−1 to −10
6. Leaky intake, suction main or gravity main from relatively unpolluted source passing through sewage-polluted water	−25 to −50
Penalties for Hazards connected with Storage and Distribution of Supply:	
7. Fishing, boating, ice-cutting, bathing, etc. on distribution or small-storage reservoirs:	
a. Fishing and boating	−2 to −10
b. Ice-cutting:	
Ice house at reservoir	−2
Ice house away from reservoirs	−1
c. Bathing	−5 to −25
8. Accessible distribution reservoirs:	
a. Open and unprotected	−5
b. Open but protected by locked fence, etc.	−2
9. Inadequate protection against polluted water leaking into pure-water reservoirs, e.g., double-storage tanks, one part for safe and other for unsafe water; leaky raw-water conduits passing through clear-water basins, etc.	−5 to −25
10. Cross-connections with polluted supplies:	
a. Single or no check valve should not be tolerated	−25 to −50
b. Double-check, F. M. type, regularly inspected	−5
c. Same as b, plus fire pump chlorinators	−3
11. Dual water-supply systems, one from a safe source and the other, for emergency use, from a sewage-polluted source	−10 to −50
12. Inadequate pressure and storage	−10
II. *Protective Measures:*	
Total credits not to exceed	30
1. Proper sanitary regulation, patrol and enforcement	4

2. Ownership of watershed, 100 percent..... (For less than 100 percent, pro-rate according to area owned)	5
3. Storage (minimum)[4].....	$2\sqrt{\text{time in weeks}}$
4. Water Purification:	
a. Chlorination.....	10
b. Pressure mechanical filters:	
With coagulation basin.....	10
Without coagulation basin.....	5
c. Gravity mechanical filters.....	15
d. Slow sand filters.....	15
e. Copper sulphate treatment, aeration, ammoniation, etc. for algae and taste control.....	5
(For combinations of the above purification methods, add individual scores, total credited not to exceed 30.)[5]	
Penalties for Failure or Inadequacy of Protective Measures:	
1. Lax enforcement of sanitary regulations, improper or inadequate regulations, irregular patrol or none.....	−2 to −4
2. Short circuits in storage reservoirs.....	−10
3. Improper operation of purification plants (not to exceed credit for plant):	
a. Inadequate dosage of chlorine.....	−10
b. Intermittent operation of chlorination apparatus, absence of duplicate equipment, etc.....	−5
c. No coagulant with mechanical filters..	−10
d. Exhaustion of chemicals or chemicals of poor quality.....	−5
e. Filters in poor condition.....	−5
f. Combustible buildings housing essential equipment.....	−5

[4] Not to exceed 20.

[5] Except that excess credits may go to offset penalties upon protective measures and a maximum credit of 35 may be allowed with double filtration and a maximum credit of 40 with double chlorination and double filtration.

4. No or inadequate laboratory control of water quality:	
a. Untreated supply	−5
b. Treated supply	−15
5. Incompetent or untrained filter operators	−15
6. Political interference	−15
III. *Quality Conditions* (Period covered not to exceed one year):	
Total credits not to exceed	30
Turbidity:[6]	
Clear	2
0 to 5 p.p.m.	1
Over 5 p.p.m.	0
Color:[6]	
Colorless	2
0 to 10 p.p.m.	1
Over 10 p.p.m.	0
Odor:[6]	
None	2
Penalty for bad odors depending on character and intensity up to	−5
Taste:[6]	
None	2
Penalty for bad taste depending on character and intensity up to	−5
Hardness:[6]	
Under 50 p.p.m.	2
50 p.p.m. to 100 p.p.m.	1
Corrosiveness:[6]	
None	1
Iron and manganese:[6]	
Less than 0.5 p.p.m.	1
Penalty for excess of 1 p.p.m.	−3
Total bacterial counts on agar at 37°C.—24 hr.:[6]	
Less than 100 per cc.	5
Over 100 per cc.	0

[6] Sum of credits for these qualities not to exceed 10.

B. coli types:[7]

Absent in 100 per cent of 10 cc. inoculations.. 20

(Present in part of 10 cc. inoculations, prorate according to percent of samples positive.)

Note: Total score not to exceed 100.

(*Sub-group: Great Lakes, connecting rivers and St. Lawrence River*)

I. *Pollution Hazards:*

Score for no hazard..... 60

Base Penalities (Population within two weeks' maximum velocity time of flow above intake in case of river, or within 30 miles of intake in case of Great Lakes (p. = population 1,000; d = distance in miles)

a. Contributing indirect population..... $-\frac{1}{2d}\sqrt{p}$

b. Contributing direct population..... $-\frac{2}{d}\sqrt{p}$

(Other pollution hazards score, where applicable, the same as for main group)

II. *Protective Measures:*

Same as for main group.

III. *Quality Conditions:*

Same as for main group.

Ground water supplies (*Main group: depth less than 200 ft.*[8])

I. *Pollution Hazards:*

Score for no hazard..... 60

Base Penalties:

1. Population (p = population):
   a. Between radius of 300 ft. and 1,000 ft..... $-\sqrt{p}$
   b. Within radius of 300 ft..... $-7\sqrt{p}$
2. Sewers, cesspools, etc., within 300 ft..... -10
3. Improperly protected top..... -5
4. Leaky casings..... -5
5. Connections with polluted surface water... -10 to -60

[7] Weekly analyses for unfiltered supplies, daily analyses for filtered supplies.

[8] "Depth" means depth of tight casing.

6. Flooding within 300 ft. by sewage-polluted surface water..... −5
7. Distribution hazards:
The same as for surface supplies.

II. *Protective Measures:*

Total credits not to exceed..... 30

1. Depth and kind of strata penetrated:
Sand under 20 ft. in depth..... 5
Sand over 20 ft. in depth..... 15
Sand and gravel under 20 ft. in depth..... 3
Sand and gravel over 20 ft. in depth..... 10
Seamy rock and limestone..... 0
Impervious layer above water-bearing stratum..... 10
2. Purification methods:
Credits and penalties the same as for surface supplies.

III. *Quality Conditions:*

The same as for surface supplies.

Ground water supplies (Sub-group: depth more than 200 ft.[8])

I. *Pollution Hazards:*

Score for no hazard..... 60
Penalties:

1. Population (p = population):
a. Within a radius of 1,000 ft..... $-2\sqrt{p}$
(Other pollution hazards the same as for main group.)

II. *Protective Measures:*

Total credits not to exceed..... 30

1. Overlying soil (same as for main group).
2. Purification methods:
Credits and penalties the same as for surface supplies.

III. *Quality Condition:*

The same as for surface supplies.

DISCUSSION

WILLIAM J. ORCHARD:[9] The essence of Mr. Chase's paper, from the viewpoint of this reviewer, is in the following sentence: "Judiciously used, comparative scores may be employed to stimulate competition in water supply betterments between rival municipalities." In voicing approval of the reasons that the author gives for the adoption of a Scoring System let us not lose sight of its tremendous power for good, of its power to bring public support, yes more than that, an irresistible public demand for water supply betterment.

Mr. Everyday Citizen is very much interested in his water supply because it plays such a vitally important part in his life. Unless, however, it has objectionable characteristics he is often disposed to vote against bond issues for improvement solely because he has not been sold on the need for such a bond issue. How differently he would regard it if the proponents of the betterment were able to say to him: "The water supply of our town rates 59. It should rate at least 85. The Town of X rates 65. The Town of Y rates 82. The Town of Z rates 91. This betterment will give our water supply a rating of 93." Such a statement would completely change his attitude from disinterested opposition to enthusiastic support. The recent health contests among the cities of the country, sponsored by the United States Chamber of Commerce, working through local chambers, and the stimulating effect that they have had upon health department activities and increased interest on the part of citizens in health matters is a splendid illustration of the potential effect of inter-community rivalry in matters like these.

All of the arguments seem to favor the adoption of a Scoring System. But, to be effective it must be universally applied. It will be of the greatest value when it is generally used by all State Departments of Health in publicly rating and comparatively scoring all water supplies in its area. From that will automatically follow a nation wide rating by the United States Public Health Service and a nation wide contest by the United States Chamber of Commerce. From that must come extensive water supply betterments and water works improvements.

The essential thing is to start. With that in view the speaker urges that this Association direct its committee to bring about speedy

[9] Wallace and Tiernan, Newark, N. J.

coöperation from similar committees appointed by the American Public Health Association and by the Conference of State and Provincial Sanitary Engineers for the prompt adoption and application of a Scoring System.

Until experience makes it possible to adopt a better method it is suggested that we start with the Rating Plan proposed by Mr. Chase.

COMPARISON OF RESULTS WITH STANDARD-LACTOSE, BRILLIANT GREEN BILE AND DOMINICK-LAUTER BROTHS

By N. J. Howard[1]

Among the research work carried out at Toronto during the past year was a special study of various peptones and their effect on the rate of growth and number of bacteria developing. These studies are incomplete. In addition, work was undertaken on the isolation of the colon group in water using three primary mediums consisting of standard lactose broth, brilliant green bile broth (2 per cent bile and 1:75,000 brilliant green) and Dominick-Lauter broth. The samples examined were an equal number of raw, unchlorinated filtered and chlorinated city water. All tubes were incubated for 48 hours and positive fermenters examined at the end of 24 and 48 hours. The presumptive tubes were examined according to standard methods, and in addition all positive lactose broth and Dominick-Lauter broth tubes were planted direct to brilliant green bile broth, while positive brilliant green bile broth tubes were planted direct into lactose broth.

The results include several features of general interest. It is not the purpose of this brief article to comment on the value or merits of brilliant green bile browth as a primary or confirmatory medium. The work of Mr. H. E. Jordan and associates, and more recently the researches carried out in the State of Ohio, leave no doubt as to its value. It is, however, suggested that increased attention be given to the use of brilliant green bile broth as a confirmatory medium. In 1925, the writer in collaboration with R. E. Thompson carried out an exhaustive study at Toronto along these lines with very encouraging results. The results of more recent work is presented here to-day.

Only part of the results have been tabulated in this report. The figures are somewhat complicated and a careful study is suggested

[1]Director of Water Purification, Island Filtration Laboratories, Centre Island, Toronto, Ontario, Canada.

TABLE 1

Isolation of the colon group in water. Comparison of results obtained by the use of standard lactose broth, brilliant green bile broth and Dominick-Lauter broth

Raw Lake Ontario water

	STANDARD LACTOSE BROTH		BRILLIANT GREEN BILE BROTH		DOMINICK-LAUTER BROTH	
	Number	Percent	Number	Percent	Number	Percent
Number tubes planted	424		424		424	
Presumptive positive, 24 hours	155	36.6	155	36.6	134	31.6
Total confirmed, Standard Methods	141	33.3	147	34.7	133	31.4
Percent presumptives confirmed, Standard Methods		91.0		94.8		99.3
Total confirmed, direct method	153	36.1	155	36.6	134	31.6
Percent presumptives confirmed, direct method		98.7		100.0		100.0
Presumptive positive, 48 hours	8	1.9	5	1.2	13	3.1
Total confirmed, Standard Methods	1	0.24	1	0.24	12	2.8
Percent presumptives confirmed, Standard Methods		12.5		20.0		92.3
Total confirmed, direct method	2	0.47	2	0.47	13	3.1
Percent presumptives confirmed, direct method		25.0		40.0		100.0
Summary of all results						
Presumptive positive	163	38.4	160	37.7	147	34.7
Total confirmed, Standard Methods	142	33.5	148	34.9	145	34.2
Percent presumptives confirmed, Standard Methods		87.1		92.5		98.6
Total confirmed, direct method	155	36.6	157	37.0	147	34.7
Percent presumptives confirmed, direct method		95.1		98.1		100.0

to obtain a correct interpretation of the results. In all, 2484 tubes were planted or 828 in each medium. The total percent confirmed in lactose broth and brilliant green bile was the same namely 28.6

TABLE 2

Isolation of the colon group in water. Comparison of results obtained by the use of standard lactose broth, brilliant green bile broth and Dominick-Lauter broth

Slow sand filtered water

	STANDARD LACTOSE BROTH		BRILLIANT GREEN BILE BROTH		DOMINICK-LAUTER BROTH	
	Number	Percent	Number	Percent	Number	Percent
Number tubes planted	254		254		254	
Presumptive positive, 24 hours	103	40.6	87	34.3	67	26.4
Total confirmed, Standard Methods	93	36.6	87	34.3	67	26.4
Percent presumptives confirmed, Standard Methods		90.3		100.0		100.0
Total confirmed, direct method	101	39.8	86	33.9	67	26.4
Percent presumptives confirmed, direct method		98.1		98.9		100.0
Presumptive positive, 48 hours	4	1.6	4	1.6	22	8.7
Total confirmed, Standard Methods	1	0.39	1	0.39	21	8.3
Percent presumptives confirmed, Standard Methods		25.0		25.0		95.5
Total confirmed, direct method	1	0.39	2	0.79	21	8.3
Percent presumptives confirmed, direct method		25.0		50.0		95.5
Summary of all results						
Presumptive positive	107	42.1	91	35.8	89	35.0
Total confirmed, Standard Methods	94	37.0	88	34.6	88	34.6
Percent presumptives confirmed, Standard Methods		87.9		96.7		98.9
Total confirmed, direct method	102	40.2	88	34.6	88	34.6
Percent presumptives confirmed, direct method		95.3		96.7		98.9

while the Dominick-Lauter figures were 28.3. Out of 299 lactose broth presumptives, 237 or 79.3 percent confirmed according to standard methods. In brilliant green bile out of 253 presumptives, 237

TABLE 3

Isolation of the colon group in water. Comparison of results obtained by the use of standard lactose broth, brilliant green bile broth and Dominick-Lauter broth

Chlorinated city water

	STANDARD LACTOSE BROTH		BRILLIANT GREEN BILE BROTH		DOMINICK-LAUTER BROTH	
	Number	Percent	Number	Percent	Number	Percent
Number tubes planted	150		150		150	
Presumptive positive, 24 hours	3	2.0	1	0.7	1	0.7
Total confirmed, Standard Methods	1	0.67	1	0.67	1	0.67
Percent presumptives confirmed, Standard Methods		33.3		100.0		100.0
Total confirmed, direct method	1	0.67	1	0.67	1	0.67
Percent presumptives confirmed, direct method		33 3		100.0		100.0
Presumptive positive, 48 hours	26	17.3	1	0.7	1	0.7
Total confirmed, Standard Methods	0	0	0	0	0	0
Percent presumptives confirmed, Standard Methods		0		0		0
Total confirmed, direct method	1	0.67	0	0	0	0
Percent presumptives confirmed, direct method		3.8		0		0
Summary of all results						
Presumptive positive	29	19.3	2	1.3	2	1.3
Total confirmed, Standard Methods	1	0.67	1	0.67	1	0.67
Percent presumptives confirmed, Standard Methods		3.4		50.0		50.0
Total confirmed, direct method	2	1.33	1	0.67	1	0.67
Percent presumptives confirmed, direct method		6.9		50.0		50.0

or 93.7 per cent confirmed, while in Dominick-Lauter broth out of 238 presumptives 234 or 98.3 per cent confirmed. When the presumptive lactose broth and Dominick-Lauter tubes were planted

TABLE 4

Isolation of the colon group in water. Comparison of results obtained by the use of standard lactose broth, brilliant green bile broth and Dominick-Lauter broth

Combined results, raw, filtered and chlorinated water

	STANDARD LACTOSE BROTH		BRILLIANT GREEN BILE BROTH		DOMINICK-LAUTER BROTH	
	Number	Percent	Number	Percent	Number	Percent
Number tubes planted	828		828		828	
Presumptive positive, 24 hours	261	31.5	243	29.3	202	24.4
Total confirmed, Standard Methods	235	28.4	235	28.4	201	24.3
Percent presumptives confirmed, Standard Methods		90.0		96.7		99.5
Total confirmed, direct method	255	30.8	242	29.2	202	24.4
Percent presumptives confirmed, direct method		97.7		99.6		100.0
Presumptive positive, 48 hours	38	4.6	10	1.2	36	4.3
Total confirmed, Standard Methods	2	0.24	2	0.24	33	4.0
Percent presumptives confirmed, Standard Methods		5.3		20.0		91.7
Total confirmed, direct method	4	0.48	4	0.48	34	4.1
Percent presumptives confirmed, direct method		10.5		40.0		94.4
Summary of all results						
Presumptive positive	299	36.1	253	30.6	238	28.7
Total confirmed, Standard Methods	237	28.6	237	28.6	234	28.3
Percent presumptives confirmed, Standard Methods		79.3		93.7		98.3
Total confirmed, direct method	259	31.3	246	29.7	236	28.5
Percent presumptives confirmed, direct method		86.6		97.2		99.2

Note: *Direct method.* In this procedure all positive presumptive lactose broth and Dominick-Lauter broth tubes, were planted direct into brilliant green bile broth. Positive presumptive brilliant green bile broth tubes were planted direct into lactose broth. These were in addition to Standard Method confirmation tests.

direct into brilliant green bile broth, 86.6 and 99.2 per cent confirmed as against 79.3 and 98.3 per cent according to standard methods. The brilliant green bile broth presumptive when planted direct into lactose broth gave 97.2 as against 93.7 per cent when confirmed by standard methods.

Briefly, in raw water samples, brilliant green bile produced 1.4 percent more confirmed results than lactose broth, and Dominick-Lauter broth gave 0.7 percent more than lactose broth. In filtered water, the use of lactose broth resulted in 37 per cent recovery. Brilliant green bile and Dominick-Lauter broth each gave 34.6 percent, a difference of 2.4 percent in favor of lactose broth. In chlorinated water, three 24-hour and twenty-six 48-hour tubes fermented out of which only one or 3.4 percent confirmed when lactose broth was used. Brilliant green bile and Dominick-Lauter broth each produced one presumptive in 24 hours and one in 48 hours, of which one or 50 percent confirmed in each medium.

In raw and chlorinated water lactose broth would appear to show no advantage over brilliant green bile and only slight advantage over Dominick-Lauter browth. In unchlorinated filtered water lactose broth showed a recovery of 2.4 per cent greater than either of the other mediums. When the direct planting method from the primary medium was used a greater recovery occurred from all three media. In this procedure, a great saving of time and labor results, with an apparent increase in recovery of the B. coli group. The Dominick-Lauter medium showed a remarkably high percentage of recovery, which in some cases reached 100 percent. In the type of water examined this medium was markedly slower than either lactose broth or brilliant green bile broth and is more difficult to prepare than either of these media. Regardless of the high percent of recovery this medium would seem to possess little advantage over brilliant green bile broth. The success of Dominick-Lauter broth is dependent upon the use of dyes of established suitability without which the medium may be of limited value.

In our study, our thanks are due to Mr. H. G. Dunham of the Digestive Ferments Co. for supplying a quantity of erythrosine. The researches are being continued on a larger scale during the year. Buffered media without the addition of inhibitory reagents will also be used. If the present-day seemingly unjustified prejudice against the use of inhibitory reagents in primary media is to be overcome, it would seem necessary to work along these lines. Mr. R. E. Thompson has been associated with the writer on the whole of the work undertaken in the laboratory during the year.

CARE OF PUBLIC WATER SUPPLIES

By Carl Wilson[1]

Perhaps a paper on this subject could not open more effectively than with a quotation from one of the most recent additions to the bibliography of water-borne disease, wherein is laid down a direct challenge to you and to me which we dare not ignore. Let me read from "The Significance of Water-borne Typhoid Fever Outbreaks"[2] by Abel Wolman and A. E. Gorman:

"One of the most significant findings in this study is the fact that, in the United States and to a lesser degree in Canada, waterborne outbreaks during the past decade have been due as much to laxity in handling water from its source to the consumer as to the use of water from polluted sources. Certainly no stronger indictment of our methods of supervision and control is necessary to focus serious attention on this problem by health and water works officials. In the United States, excluding the 14 outbreaks for which the cause is unknown, the records show that 102 of a total of 228 outbreaks, or 44.7 percent, were due to contamination occurring in the collection, delivery, storage or laxity in treatment. For Canada this percentage is 40.5.

"The indications are that, in a high percentage of these cases, outbreaks could have been prevented by adequate inspection and control measures. Perhaps the most obvious cases reported is that of one of America's largest cities, Detroit, where, in spite of the fact that the city was equipped with one of the most modern rapid sand filtration plants, in February, 1926, an outbreak of waterborne dysentery occurred accounting for at least 45,000 to 50,000 cases."

"It is, of course, a recognized fact that the use of surface or ground waters subject to pollution and without adequate purification treatment is an invitation to waterborne outbreaks. But when water is known to be polluted and necessary purification devices have been provided, is it not a serious reflection on both the water works and health officials procedures that outbreaks of typhoid fever and dysentery should occur? Two thousand and fifty-five cases of typhoid fever in the United States occurred because of pollution of an apparently safe water as it was being distributed to the consumer. Of these, 1995, or over 95 percent, were due to unprotected cross connections with polluted water supplies. Among these were the disastrous outbreaks at Winona Lake and Fort Wayne, Indiana, and Bloomington, Illinois."

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[1] Director of Sanitation, Department of Water and Power, Los Angeles, California.

[2] Journal, February, 1931, page 160.

"Inadequate control over purification methods accounted for 1925 cases. Of these 395 were cases where treatment consisted of filtration and chlorination—a combination amply capable of giving protection against heavily polluted water if efficiently operated. Inadequate chlorination, where this treatment was the sole health protection, was responsible for 985 cases. This clearly indicates lack of proper control measures. With proper control measures of simple character all of these cases could have been prevented."

If we are to accept this challenge, let us consider for a moment how we should set about the task of caring for public water supplies so successfully as to make polluted water in service almost impossible.

KIND OF POLLUTION TO BE GUARDED AGAINST

The important water-borne diseases in America are typhoid fever and dysentery of bacterial or amoebic origin. Our present knowledge of these diseases and the organisms causing them is that they are peculiar to man and the anthropoid apes. No other wild or domestic animals may have them, or harbor the causative organisms. Therefore, we conclude that the only dangerous type of pollution of water is that by body discharges from man himself. Pollution by animals, although offensive and in every other way objectionable, cannot cause disease. Based upon this conclusion an immense amount of thought and effort has been devoted to means for differentiating between pollution by man and by animals, but so far the results attained do not justify us in being too liberal in our interpretation if we are to be safe. Furthermore, while the diseases already mentioned are those generally transmitted by water, others may be added to the list under peculiar circumstances, particularly where water is taken from a surface supply and not stored in reservoirs before use. Anthrax is typical of such diseases, and should always be borne in mind in a cattle raising country, if anthrax is known to exist, or to have existed extensively within a decade, for its spores may remain virulent in or upon the soil for as much as ten years, and if washed into a stream immediately above a water works intake, they might cause the disease in persons drinking the water, although a reservoir of sufficient capacity intervening between the point of pollution and of use would remove the spores through sedimentation. Under such circumstances, of course, animal pollution is as important as contamination by sewage.

Our search, then, is for fecal pollution, and the best index at present available is the colon bacillus. These bacteria are so small that 25,000 of them placed end to end would measure only an inch, yet

the bacterial method of examination of water is so sensitive that only one or two of these bacteria in a tumbler of water could not fail of discovery, and their numbers would be accurately estimated by the usual laboratory methods!

The first half of the problem is solved when we know what kind of pollution we have to detect, and how to measure its amount when present. How shall we apply this knowledge so that we may finish our task by making water safe when it reaches the consumer's tap?

Naturally preventive measures are of greatest importance, and this means careful inspection of catchment areas, impounding reservoirs, streams, intakes, conduits and treatment plants and devices. A good sanitary inspector must have the instinct of Sherlock Holmes, for he is constantly encountering new conditions and combinations of circumstances with which he is not familiar, and frequently he must see through stone walls, and underground. He should be familiar with water works practices, and with the structures and devices used in conducting, handling, storing and treating water, and if he has to contend with cross connections he should know much about plumbing practice and the habits of stationary engineers, the ways of some of whom are as dark and devious as the basements and subterranean passageways in which are to be found their boilers, filters, softeners and laberinth of pipes.

He must be conversant with the sanitary code of his State and City in order that he may not only always comply with its provisions, but know when, where and how to invoke the aid of legal measures to secure necessary protection. The successful inspector must keep abreast of his work by reading technical books and magazines, by membership in public health organizations and by association with others engaged in the same work.

The sanitary inspection of field conditions reveals much, and careful records of such inspections should be kept for future reference. Bacterial samples taken at carefully selected points are of value, but the interpretation of laboratory findings must be correlated with the inspection, and the bacterial data should be recorded with the field notes. Where objectionable pollution is discovered appropriate means of abatement will of course be adopted.

A RAINY DAY FOR INSPECTIONS

A beautiful spring morning when all Nature is calling us out of doors to enjoy the sunshine upon the green hills is generally the incentive to make that inspection trip, but if such investigations can

be made only at rare intervals because of a scanty budget allowance, let me warn you, such a day is not propitious. To be sure your report would be flattering to their community and look well in print, but leave the production of such literature to your Chamber of Commerce. You are of most use to humanity when you find bad conditions, and in this you are most likely to be successful when heavy storms with accompanying surface wash betray sources of pollution, causing sewers to run in flood, and reservoirs and conduits to betray their weaknesses. Moreover, at such times careless plant operators are most likely to be caught off their guard. Chlorination is most severely taxed, and filters as well, and then is your best chance to render real service, both in actually securing protection, and in friendly coöperation with plant operators, teaching them the pitfalls into which they may be slipping, and showing them how and what to avoid. You may be uncomfortable, cold and wet; you may pay the extreme sacrifice, but you will be loyal to your calling and those whom you seek to protect, and although you may be unsung you will be proving that you are made of the stuff of which heroes are made. Wet days are hey days to sanitary inspectors.

SAMPLING

The taking of samples is an important matter, demanding the exercise of great care that they may be truly representative. Naturally they must not be polluted by the operator in taking, or in transit. Never use any bottles except those which have been sterilized in the laboratory, and so protected that they are still sterile when used. Do not allow anything to touch the lip of the bottle or that part of the stopper which enters the bottle, for to do so is to invite failure. Time the taking of samples so that they will reach the laboratory in the shortest possible time, without having to wait hours at the express office before train departure. Always ice samples heavily, and notify the laboratory of their shipment so no delay may occur in examination after arrival at destination. Remember that laboratory work is costly, and consumes much time, so do not take more samples than are needed, and select the sampling points carefully to give the desired information.

CHLORINATION CHECKS

When water is found to be actually polluted, or so exposed to pollution as to be at least potentially unsafe, the safe procedure is to insist upon chlorination. All surface waters should be continu-

ously chlorinated and many well waters which have never shown contamination are protected in the same manner as an insurance against unexpected accident. A continuous record of plant operation should be kept, showing date and hour of reading, the amount of water flowing, weight of chlorine tanks, meter readings when available, the intended dosage and the actual applied dosage, as well as the time of changing tanks. Last, but particularly important, ortho-tolidin readings to show the residual chlorine twenty minutes after dosage should be made regularly and made a part of the record. Protection is guaranteed not by the amount of chlorine applied, but by the residual chlorine still present after twenty to thirty minutes contact with the water. This test can be made frequently by any plant operator, and is worth more than a dozen laboratory reports, although the careful operator will want bacterial proof that his water has been safe. The location of a chlorination plant is of great importance. It must intervene between the consumers and the last point where pollution could occur, and, if possible, half an hour's flow above the point of use. The location should be easy of access to encourage frequent attention, and preferably where meters, weirs, or other devices afford information as to the amount of water to be treated.

THE OPERATOR IS THE KEY-MAN

A volume might be written about the care of water supplies, but in the end we are dependent upon "the man on the job." If we can not only educate them in methods and technic, but also inspire them with a keen realization of their responsibilities, and a wholesome pride in performance, then we will have continuously safe water at all times. It seems to me the sanitary inspector should make every effort to instruct and make enthusiastic the plant superintendent and operator, and to inspire these men with the vision of their duties and their opportunities to serve their fellow men. An inspector must be a detective and a police officer, but most of all he should be a friend and a teacher.

The biblical mottoes of our childhood seem to have gone out of style in late years, but I often wonder if much good would not result if one of these old fashioned devices could be hung over the door of every chlorination plant, quoting Genesis 4:9, "Am I my brother's keeper?" Serious contemplation of this question would impel the kind of self-sacrificing duty which alone can make public water supplies safe.

REPORT OF THE COMMITTEE ON FILTERING MATERIALS

At the 1931 meeting of the American Water Works Association, the Water Works Practice Committee authorized the chairman of the water purification division to appoint a committee on filtering materials. The committee that was appointed consisted of John R. Baylis, chairman, Frank W. Herring, secretary, W. M. Wallace and James W. Armstrong.

The purpose of the committee is to study filtering materials and to encourage the development of scientific knowledge of the behavior of these materials when used in water purification plants. This will include a study of sand, gravel and any other material which may be used for straining the suspended matter from water. The study will be largely confined to rapid sand filters for the present, although slow sand filters may be given consideration at a later date. The committee itself will not engage in research work on filtering materials, although individual members of the committee may do so. At present research work on filtering materials is being directed by three members of the committee, but this is being done by each member individually and is not considered a part of the committee work.

Our duties also involve the encouragement of research work in filter plants over the country, coöperation with investigators working on the more fundamental and scientific phases of the subject, correlation and coördination of the results of such experiments, studying the records of such experimental work as has been done in the past, and making known to the members of the Association from time to time the current development in such knowledge. The committee will call attention to the need for information along certain lines and will encourage those who are in a position to do research work to take up the subjects when more information is needed. There are several laboratories and filtration plants throughout the United States equipped for conducting research work and the committee does not believe that progress would be much faster if it was directing the work in some specially equipped laboratory or filtration plant.

It is not the intention of the committee to anticipate in any way

the publication of reports by independent investigators or to attempt to exercise any degree of supervision over their work. It is hoped, however, that by acting as a clearing house for information of this nature, the committee can perform valuable service in avoiding duplication of effort and in obtaining collateral data in cases where such data would be relatively easy to obtain, but not provided for in the original program.

COÖPERATION WITH THE COMMITTEE ON FILTERING MATERIALS FROM THE AMERICAN SOCIETY OF CIVIL ENGINEERS

The committee on filtering materials from the Sanitary Engineering Section of the American Society of Civil Engineers has underway a fairly extensive investigation of filter sand. This work is being directed by Mr. James W. Armstrong who is also a member of our committee. Their work should bring forth information of great value, and as this committee has been at work for some time, our committee has not encouraged additional work which will cover the same ground as the work of the A. S. C. E. Committee. Such research work that we may do, either individually or as members of the Water Works Committee, which comes within the work outlined by the A. S. C. E. Committee, will be turned over to this committee so that they will receive credit for the work. Two members of our committee (Armstrong and Baylis), already are conducting experiments for the A. S. C. E. Committee.

INVESTIGATIONS UNDERWAY AT THE PRESENT TIME

Information on the investigations now underway have come to the knowledge of the committee from four different sources. The program of coöperative research now being carried on by the Committee of the American Society of Civil Engineers will be discussed by Mr. Armstrong following the presentation of this report. Some of the experiments conducted at the Chicago Experimental Filter Plant will also be described by Mr. Baylis. In addition to this, work on filtering materials is under way at the Detroit Filtration Plant and at the Harvard Engineering School.

Work at Detroit

At the Detroit Filter Plant, Roberts Hulbert and Douglas Feben have carried on experiments for nearly a year on the hydraulic properties of filter sand. The apparatus used consists of a series of six

glass tube filters, 6 inches in diameter, each filled with filter sand and gravel. In each tube the sand used is as near complete uniformity in size as has been possible to obtain by careful sieving.

An attempt is being made to determine quantitatively for clear water the effect upon loss of head of the rate of filtration, temperature of water, depth of sand bed and size of filter sand. It has been determined so far that loss of head is directly proportional to depth of filter bed and to rate of filtration, in complete agreement with the theory on the subject. Water temperature has been found to have a marked influence upon hydraulic losses through the sand bed, the loss of head increasing rapidly as the temperature falls. Some of the earlier data obtained in this study are now viewed as not having sufficient accuracy because the temperature readings were not made with the proper degree of precision. Further observations of increasing water temperatures to be made during the coming period will serve to complete an accurate body of data. Sand size has also been found to have a great influence upon hydraulic losses. Within the limits of accuracy of the present data, it has been possible to formulate tentatively the effect of the four factors involved, bearing in mind that the sand size referred to is based upon a sand bed having a high degree of uniformity.

The technique used for studying the effect of non-uniformity in the sand bed is as follows: A sand bed of known depth and consisting of material of known grading is used to filter water at a known rate and temperature, and the actual loss of head is measured. Using the formula derived from experiments on uniform sand a calculation is made to determine the equivalent uniform size of the graded sand. This equivalent size is then studied in connection with the grading curve of the sand under test in an attempt to discover a rational factor of correlation. It has been observed further that angularity of the sand also has an effect upon hydraulic losses, the more angular sand creating a lower loss of head.

Work at Harvard Engineering School

For two years the hydraulic characteristics of sand, stratified and non-stratified, as they occur in rapid sand and slow sand filters have been under investigation at the Harvard Engineering School under the direction of Professor Gordon M. Fair. Professor Fair writes:

"In our work we are covering a considerable range of size, uniformity and particle shape. We have developed a rational equation for the flow of water through sand based upon the general dimensionless formula for flow through circular pipes. We have tested by means of carefully controlled laboratory experiments the individual relationship brought forth by this rational equation, and have determined the functions of particle size, uniformity and shape as well as packing which enter into the equation.

"Determinations of particle size have been made by direct measurement under the microscope. We are describing the size and uniformity by means of the geometric mean of the measured sizes and their geometric standard deviation. We are describing particle shapes by a numerical shape factor, which together with size and uniformity gives us the physical parameters which we consider to be of primary importance in evaluating hydraulic characteristics of sand. These are surface area and volume of material. We hope to publish our studies within the next six months."

Work at the Chicago Experimental Filtration Plant

A fairly extensive investigation of filter sand is under way at the Chicago Experimental Filtration Plant. Most of the tests are being conducted in glass tube filters. Figures 1, 2 and 3 illustrate how the glass tube filters are set up. The water is usually taken from the settling basins, although it may be taken from any point of the plant if desired. For all experiments, except the tests on filtration rates, the filters are operated at a rate of 2 gallons per square foot per minute.

In one battery of filters the sand varies widely in size, but the grains in each particular filter are as nearly uniform as is possible to obtain by sieving. To illustrate, the sand in one filter is material which was retained on a 10-mesh sieve after passing an 8-mesh sieve; the sand in the second filter was retained on a 16-mesh sieve after passing the 10-mesh sieve, and so on down to a filter containing sand retained on a 60-mesh sieve after passing a 50-mesh sieve.

With this battery of filters, information is being obtained on the effect of the size of the sand upon length of filter run, its effect upon the removal of coagulated matter and the rate of wash necessary to properly suspend the sand during washing periods. Various depths of sand are being tested with another battery of the tubes. Also another set of the tubes are filled with sand furnished by Mr. Armstrong, and the tests are being conducted in accordance with the procedure outlined by Mr. Armstrong. Considerable data are already available, but it is believed advisable not to try to interpret the results before the tests have been completed.

EFFECT OF THE SIZE OF SAND UPON THE LENGTHS OF FILTER RUNS
(BAYLIS)

The curves in figure 4 show the effect of the size of the sand upon the lengths of filter runs, using the battery of tubes containing sand

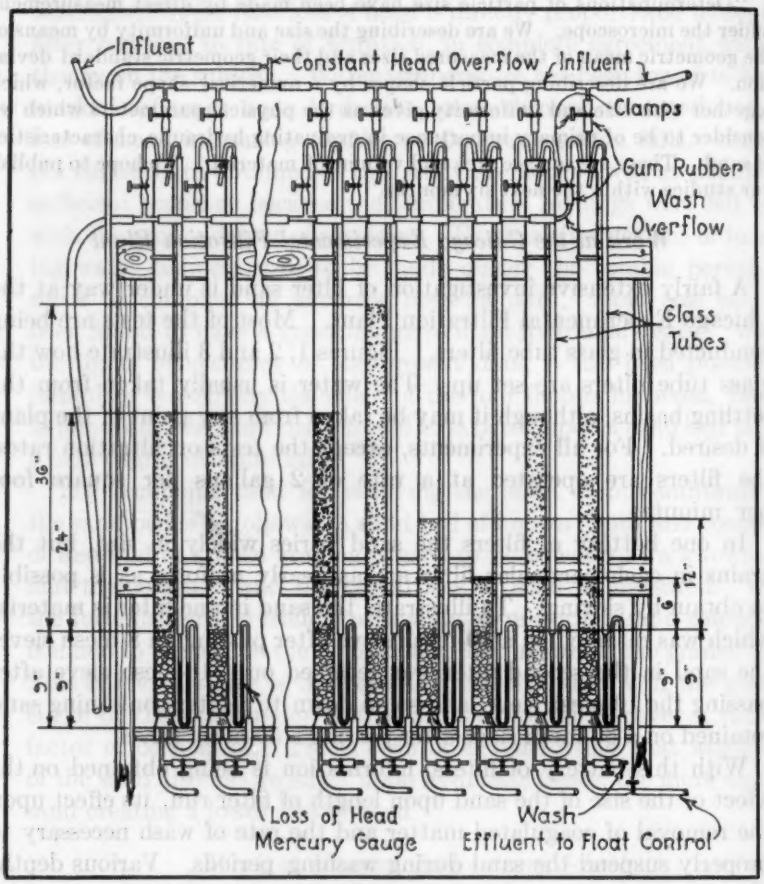

FIG. 1. BATTERY OF GLASS TUBE FILTERS

of uniform size in each unit. One of the curves is for winter conditions when the runs on a filter containing sand of 0.5 mm. for the 10 percent size are fairly long, and the other curve is for spring conditions when microörganisms are fairly abundant. The curves do not

represent extreme conditions and there may be times in many filtration plants in which the rate of increase in loss of head will be greater or lesser than the curves shown. On the basis of information col-

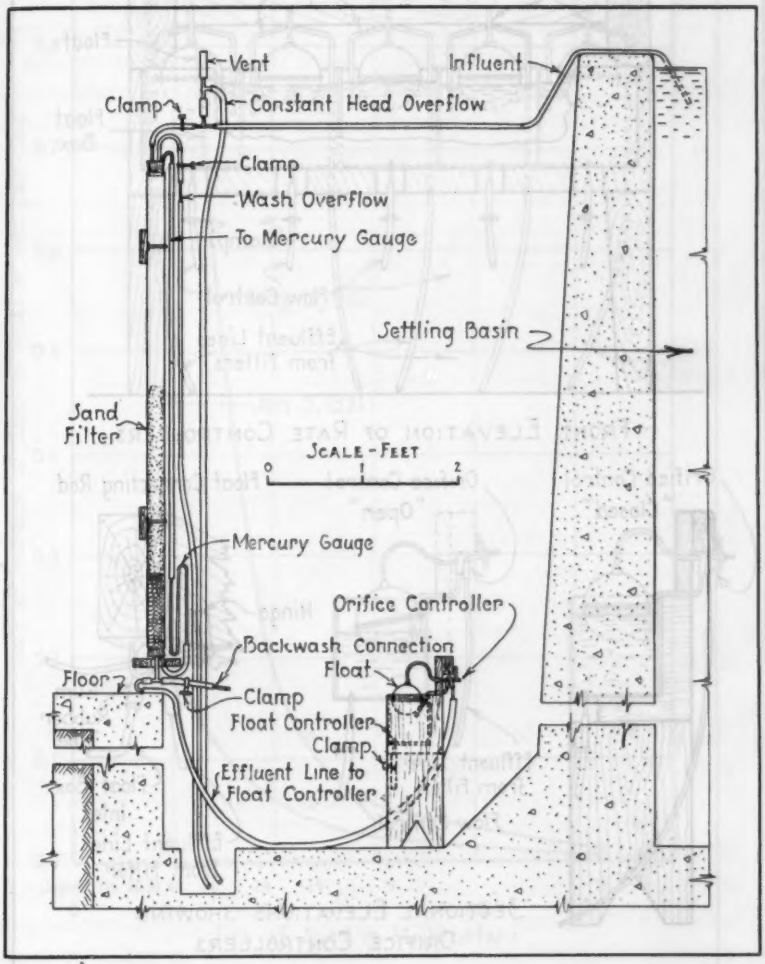

FIG. 2. SECTIONAL ELEVATION OF SAND FILTER AND CONNECTION

lected from several localities, the curves shown in figure 5 have been drawn. It is believed that they are typical of what may be found throughout the United States in the average rapid sand filtration plant. For waters which have been coagulated properly and given

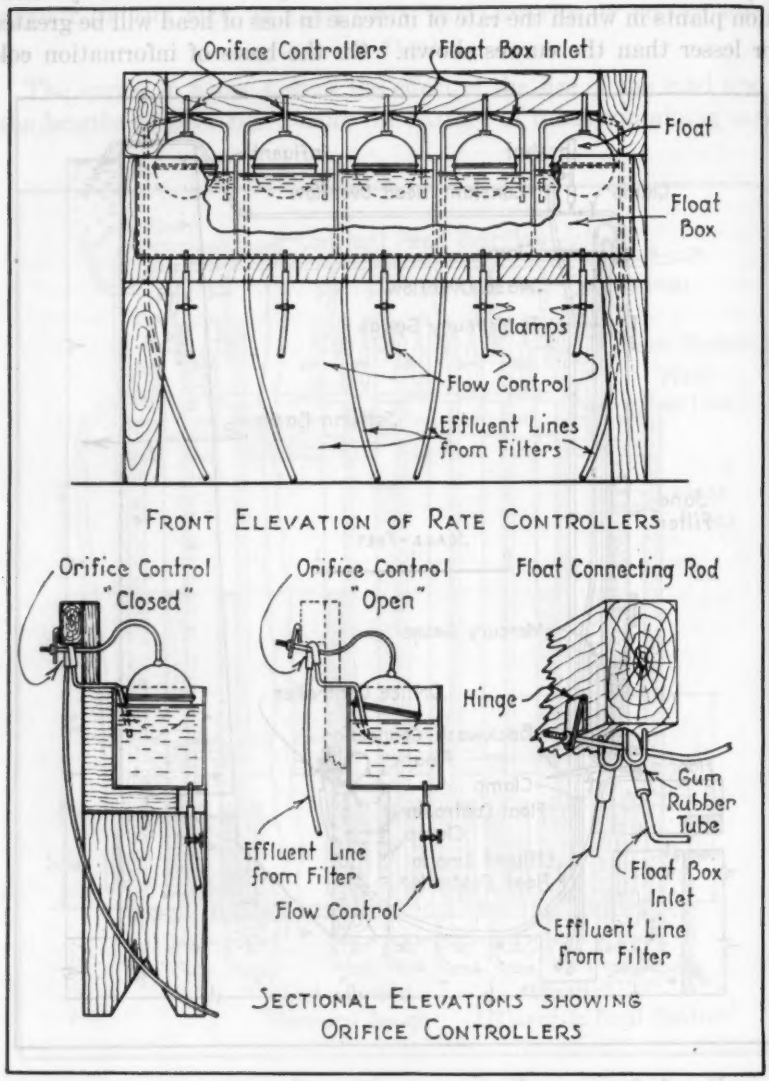

FIG. 3

several hours sedimentation and are fairly consistent in the production of short filter runs with a certain size sand, it is evident that a coarser sand should be used. If the runs are very long all the time and the

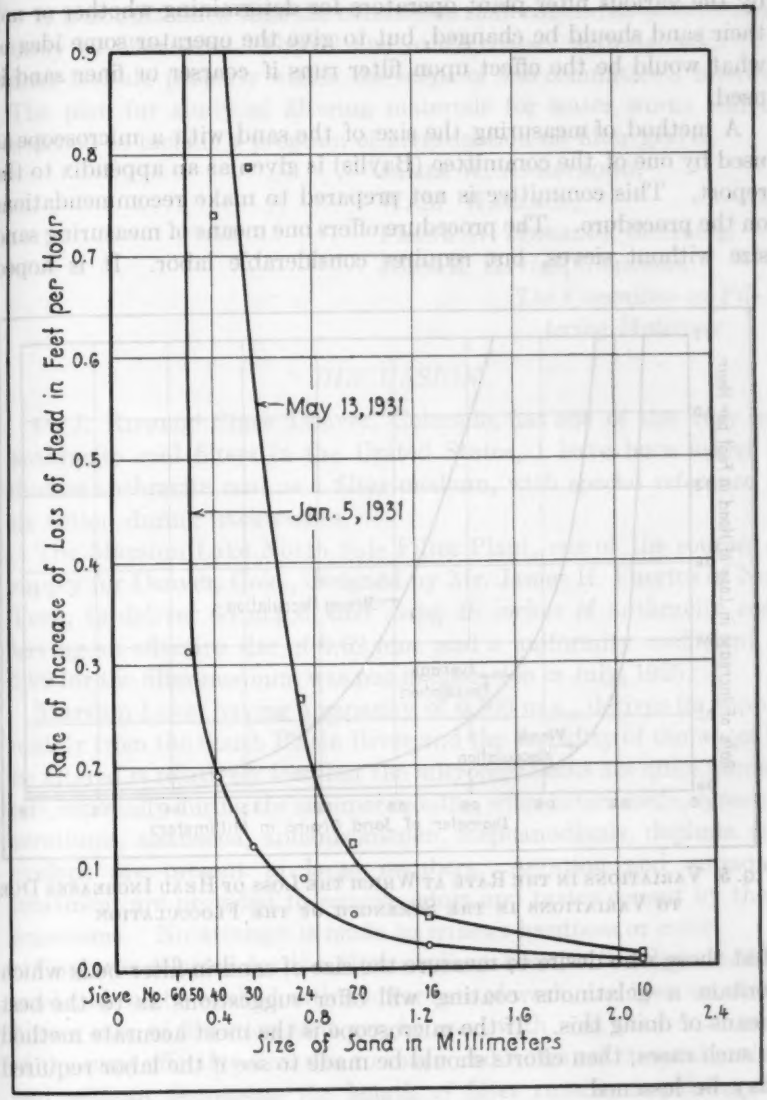

FIG. 4. RELATION BETWEEN SIZE OF SAND AND THE RATE AT WHICH THE LOSS OF HEAD INCREASES

filters pass flocculated matter at times, a finer sand probably would be better.

These few curves are given not with the idea that they may be used

by the various filter plant operators for determining whether or not their sand should be changed, but to give the operator some idea of what would be the effect upon filter runs if coarser or finer sand is used.

A method of measuring the size of the sand with a microscope as used by one of the committee (Baylis) is given as an appendix to the report. This committee is not prepared to make recommendations on the procedure. The procedure offers one means of measuring sand size without sieves, but requires considerable labor. It is hoped

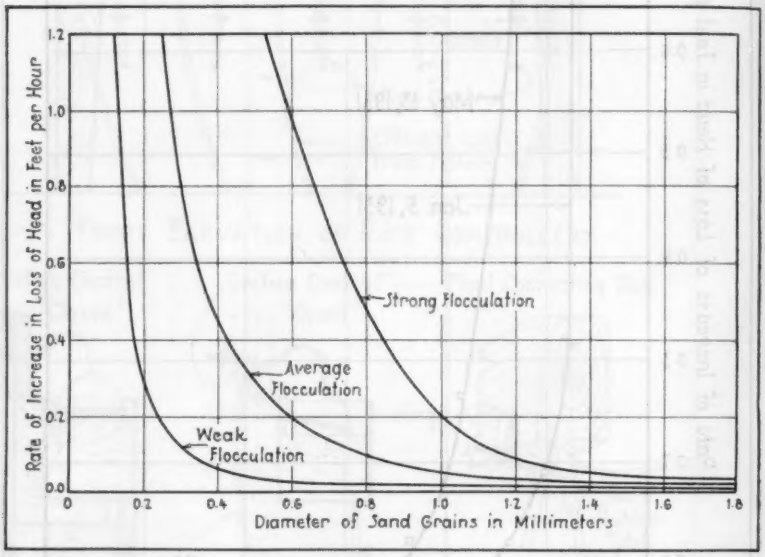

FIG. 5. VARIATIONS IN THE RATE AT WHICH THE LOSS OF HEAD INCREASES DUE TO VARIATIONS IN THE STRENGTH OF THE FLOCCULATION

that those who desire to measure the size of sand in filter beds which contain a gelatinous coating will offer suggestions as to the best means of doing this. If the microscope is the most accurate method in such cases, then efforts should be made to see if the labor required may be lessened.

During the coming year the Committee plans to make a thorough study of the existing literature on the subject of the filtration process and filter material. Such a study and the preparation of a suitable bibliography should be of the highest value in further investigational work on the subject.

It is not intended that the committee shall confine its activities entirely to filter sand. Gravel and other coarser material used in a filter bed are properly within the scope of the committee's interest. The plan for study of filtering materials for water works will be widened to include a program of investigation on filter gravel.

JAMES W. ARMSTRONG,
W. M. WALLACE,
FRANK W. HERRING, *Secretary*,
JOHN R. BAYLIS, *Chairman*.
The Committee on Filtering Materials

DISCUSSION

O. J. RIPPLE:[1] Since Denver, Colorado, has one of the very few anthracite coal filters in the United States, I have been asked to discuss anthracite coal as a filter medium, with special reference to its action during backwashes.

The Marston Lake North Side Filter Plant, one of the sources of supply for Denver, Colo., designed by Mr. James H. Fuertes of New York, to deliver 65 m.g.d. and using 48 inches of anthracite coal, having an effective size of 0.62 mm. and a uniformity coefficient of 1.99 for the filter medium, was put in operation in July, 1925.

Marston Lake, having a capacity of 6,500 m.g., derives its supply mainly from the South Platte River and the turbidity of the water to be filtered is relatively low, but the microörganisms are quite numerous, especially during the summer months, when asterionella, synedra, ceratiums, anabaena, aphanizomenon, stephanodiscus, daphnia and cyclops are present in large numbers. Aeration and ammonia treatment are provided to remove odors and tastes caused by these organisms. No attempt is made to remove hardness or color.

Coal was selected for the filter medium because it is lighter than sand and can be washed with a lower rate of wash, and because of the higher rate of filtration that can be used with coal than with sand of the same effective size, due to shape and weight of the coal grains, and without decreasing the length of filter runs or impairing the quality of the effluent. The normal rate of filtration at this plant is 167 m.g. per acre per day. When the plant was designed it was thought that the whole 48 inch depth of coal, having an effective size of 1 mm. would act as a collecting bed for practically the full

[1] Superintendent, Marston Lake, North Side Filter Plant, Denver, Colo.

depth, a theory which did not work out in the plant operation as large numbers of microörganisms and high turbidities, after relatively short filter runs, passed through the filters. The coal was removed, resized and replaced and now operates very similar to the coarser sand beds used in some filter plants.

I believe that a bed of coal 27 to 30 inches deep would be just as effective as the 48 inch depth, since most of the floc is removed in the top part of the filter bed, the only advantage of the deeper bed is that it will take a little longer for floc to begin to pass through the filter, but I do not believe that the increase in the length of filter runs justifies the cost of the extra depth.

In my opinion, air wash facilities, which are incorporated in this plant, are necessary with the coal to assist in preventing or breaking up mud balls, as the coal is too light to permit a sufficiently high rate of water wash to break up mud balls without losing coal, unless an unreasonable freeboard is used. We now have a 48 inch freeboard, which could be reduced to 36 inches without loss of coal during washes. Another reason for using air washes with coal filters is that, with the relatively low rates of water wash used with coal, together with the low specific gravity of the coal, the impact between the grains during washes is not as great as with sand at higher rates of wash. Therefore, the mud balls are not as effectively broken up and the coal grains are not as thoroughly cleaned with water washes only.

We use a 20 inch rate of wash for plain water washes and a 15 to 18 inch rate of wash following air washes, the lower rate after air washes being used to prevent loss of coal due to bubbling, which persists during the first part of the water wash, on account of residual air left in the underdrains and collecting systems, even after venting the air through special valves for that purpose. Our 20 inch rate is about the equivalent of 60 inch rate for sand of the same effective size.

We have experienced no trouble with mud balls or cracking of filter surfaces as long as we use the air for every other wash.

Data on the rise of coal during washes for various rates of wash and for two different wash water temperatures, representing the range of temperature between summer and winter, are shown in figure 6. The percent of filter bed expansion is also given. Three curves are shown for each temperature, representing the maximum, minimum and average coal sizes for filters in this plant. The difference in

effective size, between the years 1929 and 1931 is due, mainly to scraping the tops of the filters to get a larger effective size, in order to lengthen the filter runs during the summer months when micro-organisms are most numerous.

The data in table 1, relative to operation for the year 1931, may be of interest to some readers. The maximum and minimum figures given are for months, and are so given because of the rather constant quality of the raw water.

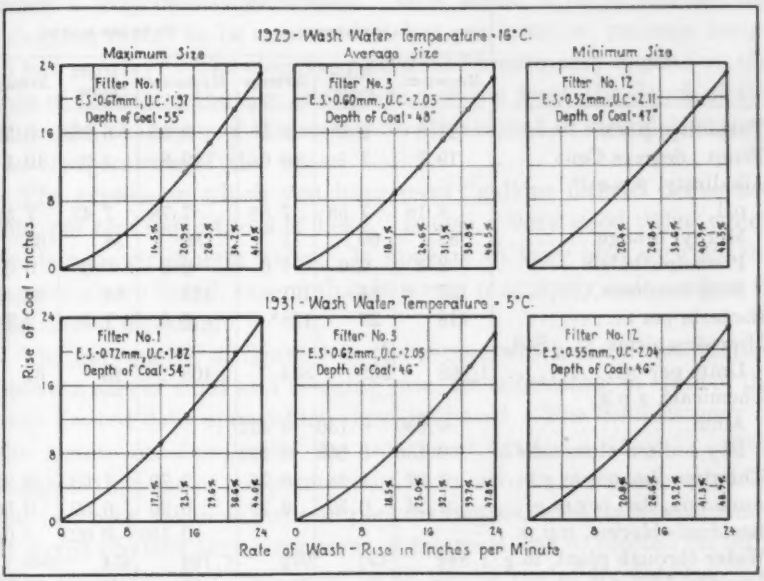

FIG. 6. MARSTON LAKE NORTH SIDE FILTERS FOR DENVER, COLORADO. FILTER MEDIA—PENNSYLVANIA ANTHRACITE COAL

Percents given are for expansion of coal beds during washes

In conclusion, I like the coal as a filter medium, very much, and believe it will be as easily and economically operated as sand, if the proper precautions are taken. From experiments conducted at this plant, in conjunction with Mr. J. W. Armstrong of Baltimore, the results of which were published in the September, 1931 issue of THE JOURNAL, it appears that the anthracite coal has a slight advantage over the sands tested at this plant, as longer filter runs with lower percents of wash water were obtained with the coal.

From the same experiments I am convinced that a combination of 12 to 15 inches of sand, approximately 0.50 mm. effective size on

the bottom and 18 to 24 inches of anthracite coal, approximately 0.83 mm. effective size, with a low uniformity coefficient, on top, can be used to advantage in many localities. The large coal on top will act as a scrubber and collect the coarser floc for its full depth and

TABLE 1

Anthracite coal as a filter medium

Operation data for 1931

	RAW WATER			FILTERED WATER		
	Maximum	Minimum	Average	Maximum	Minimum	Average
Turbidity, p.p.m.	11.3	2.0	6.5	0.95	0.26	0.53
Temp., degrees Cent.	19.7	3.1	10.0	19.8	3.2	10.1
Alkalinity, p.p.m.						
pH	8.18	7.58	7.89	7.78	7.43	7.59
Methyl orange	82	60	71	80	58	68
Phenolphthalein	2.4	0.0	1.0	0.0	0.0	0.0
Soap hardness	127	84	101	118	82	98
Bacteria per cc.	418	23	128*	6.5	1.1	2.7
Microörganisms, Std. Units per cc.	1,898	238	844	108	16	61
Chemicals, g.p.g.						
Alum	0.650	0.136	0.432			
Dry sodium aluminate	0.136	0.066	0.116			
Chlorine, lbs. per m.g.†	3.22	1.38	0.76	2.70	1.69	2.25
Ammonia, lbs. per m.g.	1.24	0.50	0.29	0.90	0.50	0.56
Residual chlorine, p.p.m.				0.130	0.023	0.080
Water through plant, m.g.	1,844	450	972	1,794	434	948
Length of filter runs, hrs.				110.5	42.0	55.9
Rate of filtration						
M.g.d. per filter				5.42	3.95	4.68
M.g.d. per acre				181	132	156
G. per sq. ft. per min.				2.90	2.11	2.50
Percent of wash water				2.55	0.82	1.36

* After chlorination.

† Prechlorination and ammonia used in the raw water part of the year only.

the sand will have to remove only the small floc, this will result in longer filter runs at a higher rate of filtration than is ordinarily used and a lower percent of wash water, with an effluent comparable with that from sand filters.

ARTHUR B. MORRILL:[2] On reading the typewritten copy of the 1932 report of the American Water Works Association committee on filtering materials, a few comments suggest themselves. The attempt of the committee to collect and analyze data as to the rate of head loss in rapid sand filters is very much worth while. One of the first difficulties is the lack of a convenient expression for the thing we are trying to evaluate.

"Rate of increase of loss of head" describes it, but that is certainly a very clumsy expression. How would it be to call this the "clogging rate," to be measured in feet per hour, or, perhaps better in feet per day? The clogging rate would presumably depend on the rate of filtration, the size and grading of the sand, the depth of the filter, the character of the raw water, the extent of coagulation, and the temperature of the water.

The expression which you have used "gallons filtered per square foot per one foot of loss of head," is also a very good value which needs a much shorter name. It appears to vary also with all the factors above listed, except that it seems practically independent of the rate of filtration.

The two curves of figure 4 in the report which show the relation between size of sand and clogging rate, are interesting in spite of the very limited data upon which they are based. The inconsistency of the results obtained with the two finest sands on May 13, 1931, suggests that one of the points must be in error or else proper account has not been taken of some important factors, such as temperature. It seems obvious that the curves of figures 4 and 5, plotted on ordinary cross-section paper, must be of the nature of hyperbolas. As the size of sand approaches zero the clogging rate must approach infinity and as the size of sand gets larger and larger the clogging rate must approach zero. With such curves, their true nature and trend can be much more clearly shown on logarithmic paper. Figure 7 shows the data of figures 4 and 5, replotted in this way.

The six points representing the data of January 5, 1931 fall fairly well in a straight line. Very careful and extensive data might show, of course, that the true graphs on log paper are not straight, but curved in some way. The reciprocal of the slope of the curve is

[2] Assistant Engineer of Filtration, Department of Water Supply, Detroit, Mich.

minus 2.09 which means that the clogging rate varies inversely as the 2.09 power of the sand diameter.

The data for May 13, 1931, are less satisfactory, particularly because of the inconsistent results obtained with the two finest sands. This set of observations suggests an exponent in the equation of 2.70 instead of 2.09.

The three narrow curved lines on figure 7 are replotted from figure 5. The parts of these lines representing clogging rates less than 0.05 feet per hour are probably not significant as the scale of the

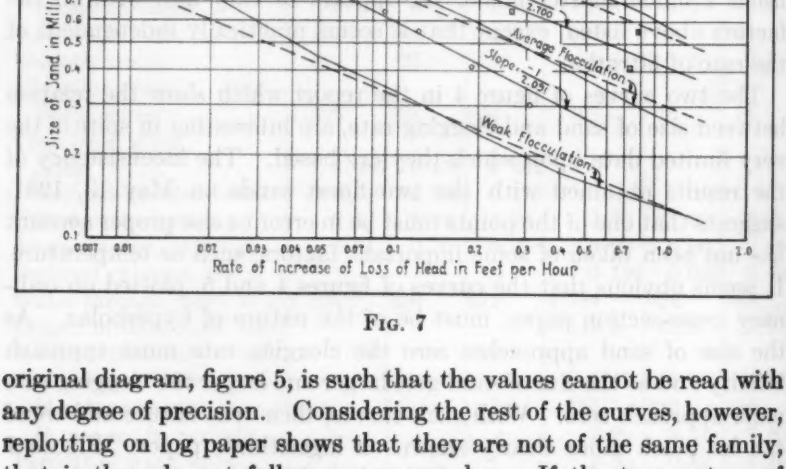

FIG. 7

original diagram, figure 5, is such that the values cannot be read with any degree of precision. Considering the rest of the curves, however, replotting on log paper shows that they are not of the same family, that is they do not follow a common law. If the true nature of these graphs is such that they would plot on log paper as straight lines, then the lines must be parallel. If they are curved then the graphs should show similarities in curvature.

The three heavy dash lines on figure 7 are drawn parallel to each other in such a way as to represent as well as possible the data from figure 5. These lines indicate that the clogging rate varies inversely as the 2.58 power of the sand diameter.

APPENDIX

MEASUREMENT OF SAND SIZE WITH THE MICROSCOPE

By John R. Baylis[3]

In some instances where the sand grains in a filter bed have become coated with gelatinous material or material which would shrink or be easily rubbed off of the sand grains when the grains are dried and sieved, estimation of the true size of the sand grains by sieves is not accurate. In how many water works such measurements would be inaccurate, we do not know, but certainly there are a few instances. The writer has given considerable thought to the measurement of sand sizes with the microscope. The procedure requires considerable work, yet it is believed to be fairly accurate and seems to be the only way the size of the grains including the coating may be accurately measured.

RELATION OF THE MIDDLE DIAMETER OF SAND GRAINS TO THE AVERAGE DIAMETER

The sand grain which is not perfectly round may be said to have three diameters: one is the longest distance through the grain; another is the shortest distance which is nearly perpendicular to the longer diameter; then there is a third diameter which is approximately perpendicular to both the longer and the shorter diameters. Sand grains lying on a horizontal surface will usually lie in a position in which the longest diameter and the middle diameter may be measured. The shortest diameter, which is in a nearly vertical position, cannot be measured unless there is some way of turning the sand grain on its side.

A number of tests were made to see if the measurement of the longest diameter or the middle diameter had some relation to the average diameter as determined by sieves, or the diameter of a sphere of the same volume. As stated, these two diameters may be measured without moving or tilting the sand grain under the microscope. It was found that the relation between the longest diameter and the average diameter was not a definite one and it would be somewhat inaccurate to attempt to use the longest diameter in estimating the average diameter of the sand grains. The middle diameter, how-

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ever, which is the shorter of the two which may be measured without tilting the grains, does show a close relation to the average diameter. Table 2 gives the results of measurements made on three different sizes of sand grains. The grains of largest diameter which passed the sieves were collected and measured; that is, the sand was sieved

TABLE 2

Comparison of the size of sand grains determined by the microscope with the size determined by sieves (December, 1928)

PASSING 10 MESH SIEVE		PASSING 20 MESH SIEVE		PASSING 24 MESH SIEVE	
Greater diameter	Middle diameter	Greater diameter	Middle diameter	Greater diameter	Middle diameter
mm.	*mm.*	*mm.*	*mm.*	*mm.*	*mm.*
2.66	2.48	1.19	1.05	1.12	0.91
3.22	2.66	1.40	1.12	1.05	0.91
2.80	2.24	1.12	1.05	1.19	0.77
2.38	2.24	1.68	1.05	1.05	0.77
2.94	2.24	1.54	1.05	0.91	0.84
3.36	2.38	1.19	1.12	1.40	0.98
3.22	2.38	1.40	0.98	1.33	0.84
2.38	2.10	1.54	1.12	0.91	0.84
		1.12	1.05	1.05	1.05
		1.54	1.05	1.12	1.05
		1.54	1.05	1.12	0.98
		1.40	0.91	0.98	0.84
		1.75	1.05	0.98	0.84
		1.26	0.98	0.98	0.70
		1.47	0.98	1.33	0.77
		2.17	0.91	1.12	0.77
		1.82	0.98	0.98	0.70
		1.26	1.12	1.05	0.91
		1.40	1.19	1.33	0.91
		0.98	0.91	1.12	0.98
Average...2.87	2.34	1.44	1.04	1.11	0.87
Diameter by sieves....	2.00		0.97		0.81

until only a few grains would pass the sieves by continued shaking of the sieves for about one minute, and it was these grains which were collected and measured. The weights of the grains were taken from the curve in figure 8. When plotted on the usual logarithmic paper containing lines very close together, the weight may be obtained very accurately. Curves should be plotted for the specific gravity of the sand, or one curve may be used and the results multi-

plied by a factor if the specific gravity is different from that of the curve. It will be explained that specific gravity is not of much importance in estimating the size of the sand.

Method of measuring the sand grains

The sand grains were placed upon a Rafter's counting cell, spread out so that none of the grains touched each other, and then the shortest diameter of the grain was measured with an eyepiece micrometer the same as is used in counting and measuring microörganisms.

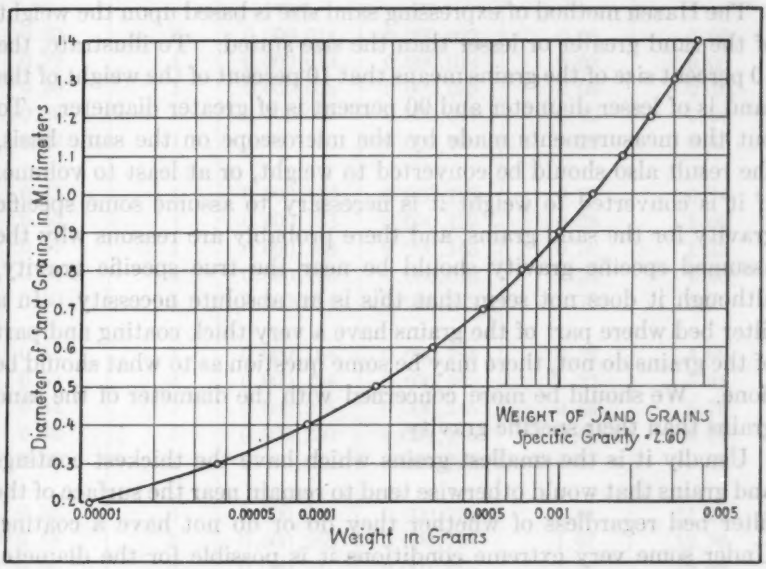

FIG. 8

The longest diameter also may be measured if desired. As previously stated, the shortest diameter of the grain as it lies upon the glass cell is the middle diameter of the grain. The average diameter of the sand grain as determined by sieving was estimated to be 1.1 times the size of the opening in the sieve. This is in accordance with Hazen's recommendation. The average diameter of the largest sand grains passing the 10 mesh sieve, which is the opening in the sieve times 1.1, was 2.0 mm. The diameter determined by the microscope, which was the average of the shortest diameters measured, was 2.34 mm., or slightly greater than the diameter determined with the sieve. The 20

mesh sieve gives an average diameter of 0.97 mm. for the largest grains passing the sieve, and the average diameter determined with the microscope was 1.04 mm. The 24 mesh sieve gives 0.81 mm. for the size determined with the sieve and 0.87 mm. for the size determined with the microscope. Clean sand with no coating was used for making the test. It is believed that the same degree of accuracy may be obtained on sand grains having a gelatinous or other soft coating. If it is necessary to make one measurement only on the sand grain with the microscope, this materially lessens the labor in determining the size of the grains.

The Hazen method of expressing sand size is based upon the weight of the sand greater or lesser than the size stated. To illustrate, the 10 percent size of the grains means that 10 percent of the weight of the sand is of lesser diameter and 90 percent is of greater diameter. To put the measurements made by the microscope on the same basis, the result also should be converted to weight, or at least to volume. If it is converted to weight it is necessary to assume some specific gravity for the sand grains, and there probably are reasons why the assumed specific gravity should be near the true specific gravity, although it does not seem that this is an absolute necessity. In a filter bed where part of the grains have a very thick coating and part of the grains do not, there may be some question as to what should be done. We should be more concerned with the diameter of the sand grains than their specific gravity.

Usually it is the smallest grains which have the thickest coatings and grains that would otherwise tend to remain near the surface of the filter bed regardless of whether they do or do not have a coating. Under some very extreme conditions it is possible for the diameter of the sand grain with its coating to be larger than some of the grains which have no coating, yet the grains without a coating remain near the bottom of the sand bed due to their higher specific gravity. For the present it is believed that specific gravity may be ignored in the determination of the size of the sand and the volume used.

COMPUTED WEIGHT OF SAND GRAINS WITH THE MICROSCOPE VERY ACCURATE

Table 3 shows that the computed weight of sand, in which the grain is assumed to be a sphere having a diameter the same as that of the middle diameter of the sand grain, is very close to the actual weight. One hundred sand grains were selected from a sample of

TABLE 3

Determination of sand size with the microscope

One hundred grains of sand were selected from a sample of sand which was from a filter bed before it was put into service. The 100 grains were weighed and then spread out on a glass slide so their middle diameter could be measured. The middle diameter is the shorter diameter observed with the microscope. The sand was from Muscatine, Iowa (April, 1932).

Specific gravity of sand 2.66. Total weight of 100 grains, 0.0474 gram.

MIDDLE DIAMETER OF SAND GRAINS	NUMBER OF GRAINS	WEIGHT OF ONE GRAIN	TOTAL WEIGHT	PERCENT OF WEIGHT	PERCENT OF THE SAMPLE OF THE DIAMETER AND SMALLER
mm.		*gram*	*gram*		
0.20	2	0.000,011	0.000,022	0.05	0.05
0.22	1	0.000,014	0.000,014	0.03	0.08
0.30	1	0.000,038	0.000,038	0.08	0.16
0.32	1	0.000,047	0.000,047	0.10	0.26
0.42	1	0.000,100	0.000,100	0.21	0.47
0.45	1	0.000,128	0.000,128	0.27	0.74
0.46	1	0.000,133	0.000,133	0.28	1.02
0.47	1	0.000,143	0.000,143	0.30	1.32
0.48	4	0.000,153	0.000,612	1.28	2.60
0.50	3	0.000,174	0.000,522	1.10	3.70
0.51	1	0.000,184	0.000,184	0.39	4.09
0.52	3	0.000,194	0.000,582	1.22	5.31
0.54	1	0.000,215	0.000,215	0.45	5.76
0.55	5	0.000,230	0.001,150	2.41	8.17
0.56	3	0.000,240	0.000,720	1.51	9.68
0.57	3	0.000,256	0.000,768	1.61	11.29
0.58	3	0.000,266	0.000,798	1.67	12.96
0.60	5	0.000,297	0.001,485	3.12	16.08
0.62	4	0.000,332	0.001,328	2.79	18.87
0.64	6	0.000,368	0.002,208	4.63	23.50
0.65	3	0.000,389	0.001,167	2.45	25.95
0.66	1	0.000,404	0.000,404	0.85	26.80
0.67	2	0.000,419	0.000,838	1.76	28.56
0.68	2	0.000,435	0.000,870	1.83	30.39
0.69	2	0.000,450	0.000,900	1.89	32.28
0.70	6	0.000,481	0.002,886	6.05	38.33
0.71	1	0.000,496	0.000,496	1.04	39.37
0.72	2	0.000,517	0.001,034	2.17	41.54
0.75	4	0.000,583	0.002,332	4.89	46.43
0.76	5	0.000,604	0.003,020	6.34	52.77

TABLE 3—*Concluded*

MIDDLE DIAMETER OF SAND GRAINS	NUMBER OF GRAINS	WEIGHT OF ONE GRAIN	TOTAL WEIGHT	PERCENT OF WEIGHT	PERCENT OF THE SAMPLE OF THE DIAMETER AND SMALLER
mm.		*gram*	*gram*		
0.77	1	0.000,624	0.000,624	1.31	54.08
0.78	1	0.000,655	0.000,655	1.37	55.45
0.80	2	0.000,711	0.001,422	2.98	58.43
0.82	3	0.000,757	0.002,271	4.76	63.19
0.83	1	0.000,777	0.000,777	1.63	64.82
0.84	1	0.000,818	0.000,818	1.72	66.54
0.85	5	0.000,839	0.004,195	8.80	75.34
0.86	1	0.000,870	0.000,870	1.83	77.17
0.89	1	0.000,972	0.000,972	2.04	79.21
0.92	2	0.001,074	0.002,148	4.50	83.71
1.04	2	0.001,586	0.003,172	6.65	90.36
1.10	1	0.001,841	0.001,841	3.86	94.22
1.25	1	0.002,762	0.002,762	5.79	100.01
Total.....	100		0.047,671	100.01	

0.047,671 − 0.0474 = 0.000,271 gram difference between actual weight and computed weight by assuming that the average diameter is approximately the middle diameter.

filter sand which had not been used in a filter bed. The grains were weighed, the middle diameter measured with a microscope and the weight was then computed. The computed weight in this instance varies less than one percent from the actual weight. This perhaps is an accident, although it is believed the variation will be less than 10 percent. Sand grains of various size were selected, although no effort was made to secure a sample that was an average of the sand. This was to measure the relation of the size of sand grain as determined by the microscope with the actual average diameter of the grain and not to determine exact size of the sand.

Table 3 also shows the steps necessary to take to convert the measurements made by the microscope into the usual method of expressing sand size as determined with sieves. The first column gives the middle diameter of the grains, which is the shortest diameter measured without tilting the sand grain. The second column shows the number of grains of the diameters given in the first column. The third and fourth columns show the weight of the grains in grams,

based upon the volume multiplied by the specific gravity. The fifth column shows the percentage of the weight for various diameters, and the sixth column is the total for each size, beginning at the smallest size and ending with 100 percent at the largest size. It is easy to estimate the amount of the sand of a size lesser or greater than some certain percentage of the total weight; that is, if it is desired

TABLE 4

Measurement of a coarse sample of Cape May sand (December, 1922)

MIDDLE DIAMETER OF SAND GRAIN	NUMBER OF GRAINS	WEIGHT OF GRAINS	PERCENT WEIGHT OF SAMPLE	PERCENT OF SAMPLE OF THE DIAMETER AND SMALLER
mm.				
0.63	2	0.000,672	0.4	0.4
0.70	1	0.000,467	0.3	0.7
0.77	9	0.005,652	3.2	3.9
0.84	10	0.008,070	4.6	8.5
0.91	16	0.016,464	9.3	17.8
0.98	9	0.011,574	6.6	24.4
1.05	16	0.025,376	14.3	38.7
1.12	11	0.021,120	12.0	50.8
1.19	6	0.014,076	8.0	58.8
1.26	11	0.030,085	17.1	75.9
1.33	4	0.012,856	7.3	83.2
1.40	3	0.011,208	6.4	89.6
1.61	1	0.005,584	3.2	92.8
2.10	1	0.012,607	7.2	100.0

1.0 percent size, 0.77 mm.
10.0 percent size, 0.91 mm.
60.0 percent size, 1.26 mm.
10.0 percent size determined by sieves 0.85 mm.

Note: There is some doubt about the sieve analysis being on the same sample of sand, although the relation of the 10 percent size on the two samples appears to be about what should be expected (J. R. B., April, 1932).

to determine the 10 percent size of the sand measured in table 3, it will be seen that it is 0.57 mm. diameter. The 60 percent size is 0.82 mm., and the 1 percent size is 0.46 mm. diameter. It should be evident from table 3 that the specific gravity is immaterial in the computation in so far as it affects determining the 10 percent size or any other size of the sand. Specific gravity might have some effect upon sand sizes determined with sieves, though for the usual filter sand it probably is of little consequence.

TABLE 5

Measurement of a sample of filter sand with the microscope (April 18, 1932)

About 300 grains of the sand were spread out upon a Rafter's counting cell and 100 grains measured. All grains passed over as the mechanical stage was moved were measured. The middle diameter in the shortest diameter observed without tilting the grains.

Sand obtained from Muscatine, Iowa. Specific gravity of sand 2.66.

MIDDLE DIAMETER OF SAND GRAINS	NUMBER OF GRAINS	WEIGHT OF ONE GRAIN	TOTAL WEIGHT	PERCENT OF WEIGHT	PERCENT OF THE SAMPLE OF THE DIAMETER AND SMALLER
mm.		*gram*	*gram*		
0.32	1	0.000,047	0.000,047	0.11	0.11
0.38	4	0.000,077	0.000,308	0.75	0.86
0.41	1	0.000,097	0.000,097	0.24	1.10
0.42	2	0.000,100	0.000,200	0.49	1.59
0.44	1	0.000,118	0.000,118	0.29	1.88
0.45	1	0.000,128	0.000,128	0.31	2.19
0.47	1	0.000,143	0.000,143	0.35	2.54
0.48	5	0.000,153	0.000,765	1.86	4.40
0.49	1	0.000,164	0.000,164	0.40	4.80
0.50	7	0.000,174	0.001,218	2.96	7.76
0.52	3	0.000,194	0.000,582	1.41	9.17
0.53	3	0.000,208	0.000,624	1.51	10.68
0.55	2	0.000,230	0.000,460	1.12	11.80
0.56	2	0.000,240	0.000,480	1.17	12.97
0.57	5	0.000,256	0.001,280	3.11	16.08
0.58	6	0.000,266	0.001,596	3.87	19.95
0.60	5	0.000,297	0.001,485	3.60	23.55
0.61	2	0.000,317	0.000,634	1.54	25.09
0.62	4	0.000,332	0.001,328	3.22	28.31
0.63	2	0.000,353	0.000,706	1.71	30.02
0.64	1	0.000,368	0.000,368	0.89	30.91
0.65	4	0.000,389	0.001,556	3.78	34.69
0.66	5	0.000,404	0.002,020	4.90	39.59
0.67	3	0.000,419	0.001,257	3.05	42.64
0.68	1	0.000,435	0.000,435	1.06	43.70
0.69	1	0.000,450	0.000,450	1.09	44.79
0.70	3	0.000,481	0.001,443	3.50	48.29
0.71	2	0.000,496	0.000,992	2.41	50.70
0.72	1	0.000,517	0.000,517	1.25	51.95
0.73	1	0.000,542	0.000,542	1.32	53.27

TABLE 5—*Concluded*

MIDDLE DIAMETER OF SAND GRAINS	NUMBER OF GRAINS	WEIGHT OF ONE GRAIN	TOTAL WEIGHT	PERCENT OF WEIGHT	PERCENT OF THE SAMPLE OF THE DIAMETER AND SMALLER
mm.		*gram*	*gram*		
0.74	1	0.000,557	0.000,557	1.35	54.62
0.76	2	0.000,604	0.001,208	2.93	57.55
0.78	2	0.000,655	0.001,310	3.18	60.73
0.79	1	0.000,675	0.000,675	1.64	62.37
0.80	2	0.000,711	0.001,422	3.45	65.82
0.81	3	0.000,737	0.002,211	5.37	71.19
0.82	1	0.000,757	0.000,757	1.84	73.03
0.84	1	0.000,818	0.000,818	1.99	75.02
0.85	1	0.000,839	0.000,839	2.04	77.06
0.86	1	0.000,870	0.000,870	2.11	79.17
0.89	1	0.000,972	0.000,972	2.36	81.53
0.95	1	0.001,176	0.001,176	2.85	84.38
1.05	1	0.001,606	0.001,606	3.90	88.28
1.20	2	0.002,414	0.004,828	11.72	100.00
	100		0.041,192	100.00	

SIZE	MILLIMETERS	
	Microscope	Sieves
1 percent	0.41	0.37
10 percent	0.53	0.50
60 percent	0.78	0.75

SAND SIZE DETERMINED BY COMPUTING FROM MIDDLE DIAMETER MAY NOT BE EXACT

Table 4 shows measurement of a sample of sand determined with the microscope nearly 10 years ago when the writer first worked out this procedure for determining sand size. Table 5 shows a recent measurement of a sample of sand and the size determined with sieves. It will be seen that the size determined with the microscope is greater than the size determined with sieves. This confirms the results given in table 2, which were made nearly 10 years ago. A recent measurement on a 30-mesh sieve shows about the same relation as the earlier measurements.

The writer does not recommend the use of the microscope for determining the size of sand which does not contain a coating, largely

from the fact that it requires much more labor than is required to sieved the sand. It is believed, however, that accurate results may be obtained in this manner. Where the sand grains have been in use in a filter bed, and contain a gelatinous coating, this probably is the only way of measuring the true size of the grains. The results given indicate some variation in the size determined by sieves and the size determined by measuring the middle diameter of the grains with a microscope. This should be investigated more fully. It may be found necessary to use a factor for the microscopical measurements to give results which will compare accurately with sieves.

MANGANESE AND ITS RELATION TO FILTERS[1]

By Perkins Boynton[1] and Lewis V. Carpenter[2]

The appearance of manganese in the surface water supplies of the bituminous coal districts has been closely allied with the development of coal mining and the subsequent acid drainage entering the streams. At the present time, this problem is limited to the small group of states in the Middle Atlantic (1) section which produce over 90 percent of the bituminous coal. Twenty-one other states have unusually large bituminous coal stocks and eventually will have to solve the problem of manganese.

OCCURRENCE

Water soluble manganese is usually absent in the bituminous coal beds. Carpenter (2) found that samples of powdered coal immersed in water showed only a trace of manganese at the end of 284 days. Surface streams which did not contain acid drainage in the same vicinity as the mines were usually free of manganese and the same was true of drippings through the roof of the coal mines. A number of samples of shale, slate and other gobbed material found along the drains did not show a sufficient quantity of it to account for the high content usually found in the drainage. Samples of slate and pyrites were pulverized and mixed with distilled water and stored at room temperature for a period of 180 days. These samples were prepared in Erlenmeyer flasks with cotton stoppers and one-half of the samples were sterilized in the autoclave for 30 minutes at 15 pounds pressure. The results are summarized in table 1.

It will be noted that all of the samples had a much higher manganese content at 180 days in the unsterilized sample. This is a partial confirmation of the biological theory of formation of acidity in the mine drainage. It has not been definitely worked out whether the solution of the manganese is accelerated by the biological growths

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or by the increased acidity of the water. Numerous tests have shown conclusively that the manganese comes into our rivers from the mine drainage. It is assumed that it occurs as a sulfate and after being neutralized by the addition of lime and according to Weston (3) absorbs oxygen, oxidizes slowly and ultimately precipitates as manganic hydrate (hydrated manganic oxide).

TABLE 1

Partial analysis of sterilized and unsterilized mixtures of shale, pyrites and water

SAMPLE	UNSTERILIZED			STERILIZED		
	pH	Acidity	Manganese	pH	Acidity	Manganese
1	2.1	564	4.0	2.8	125	0.1
2	2.2	724	5.4	2.8	290	0.2
3	2.5	635	6.1	3.5	392	0.2
4	3.5	164	0.3	5.0	4.0	0

TABLE 2

Manganese content of streams polluted with acid mine drainage

DATE	STREAM	pH	PARTS PER MILLION	
			Hardness	Manganese
November, 1929	Tygarts Valley	2.8	524	7.0
August, 1929	West Fork, Weston, W. Va.	6.5	65	2.1
September, 1929	Monongahela River, Morgantown, W. Va.	3.8	382	1.8
October, 1930	Monongahela River, Morgantown, W. Va.	6.6	190	0.2
October, 1930	West Fork, Clarksburg, W. Va.	4.5	278	2.0
July, 1930	Brown's Creek, Clarksburg, W. Va.			12.0
July, 1930	Scotts Run, Morgantown, W. Va.	2.0		31.0

Table 3 gives the manganese content of the raw water at a number of plants where the sand has become coated with manganese. In addition, the sand in the filters at Olney, Ill., Princeton, Ind., Minneapolis, Minn., Barberton, Ohio, Pulaski, Shelbyville and Columbia, Tenn., have become badly encrusted. At Carbondale, Ill., a number of complaints were made by consumers on two different occasions.

Both occurrences were immediately after the reservoir had been given a large dose of copper sulfate to kill algae. The filter sand was coated with manganese which seemed to dissolve during this period. During the summer of 1931 the raw water at Staunton, Ill., had a manganese content of 6 p.p.m. and no dissolved oxygen. Their trouble was corrected by raising the inlet pipe.

Most of the West Virginia waterworks which have been affected with manganese are troubled with the sand caking to such an extent that ordinary washing will not break up the sand and underwater cracking occurs frequently. In one plant it is common practice to rake the filter before applying the wash water. Mud balls are quite

TABLE 3

Manganese content of some water supplies where sand has become incrusted

LOCALITY	MANGANESE
	p.p.m.
Carbondale, Ill	0–1
Staunton, Ill	0–6
Salem, Ill	
Murphysboro, Ill	0–6
Mount Vernon, Ill	1
Baltimore, Md	0–1.4
Luke, Md	0.1–0.8
East Liverpool, Md	0.22
Pittsburgh, Pa	0.10
Clarksburg, W. Va	0–4.5
Weston, W. Va	0–2.1
Fairmont, W. Va	0–7
Morgantown, W. Va	0.1–3.0

common and while it is not quite fair to lay the blame of all of these on the manganese, it probably plays an important part. Patrick, at Luke, Md., treating a river water polluted with mine waste waters had difficulty with the manganese settling out in the mains. Caustic soda and alum are now used as a coagulant with the filter effluent maintained at a pH of 7.2 and the manganese remained in solution. He found that overaeration of the raw water complicated the treatment due to the formation of the carbonate. This plant primarily furnishes water for paper manufacture, although a small part of the supply is used for domestic purposes and the manganese in solution does not seem to affect the paper manufacture.

It is known that a manganese bearing water will increase the size of the sand grains in a rapid sand filter. Zaffle (4) and Egger (5) claim that a sand filter that has become impregnated with manganese will become very effective in the removal of both iron and manganese, acting as a catalyzer. Experience at West Virginia plants bears out this belief, provided the pH of the water is held above 8.2. The question that presents itself now is, how large a deposit shall be allowed to accumulate on the sand before it will lose its efficiency for its normal functions? Table 4 gives the partial analysis of the sand from several filters.

The Weston plant renewed the sand about two years ago and the Clarksburg plant during the present winter. Morgantown removes the sand from one filter every year and cleans it with a weak solution

TABLE 4

Partial analyses of filter sands

LOCATION	YEARS IN FILTER	EFFECTIVE SIZE	UNIFORMITY COEFFICIENT	PERCENT	
				Manganese	Total incrustration
		mm.			
Weston, W. Va	10	0.51	1.53	0.50	1.06
Clarksburg, W. Va	20	0.65	1.38	1.03	2.66
Morgantown, W. Va	20	0.63	1.65	0.51	1.08
Barberton, O	4	0.57	1.28		5.50

of acid. Patrick at Luke, Md., found SO_2 water very satisfactory for removing manganese, but a majority of plants have found it more economical to replace with new sand.

EFFECT ON OPERATION

The prevalence of manganese in the waters of the rivers of this section of the country is exerting a marked effect upon the operation of purification plants. Whereas, formerly the operation of a filter plant was directed towards the removal of turbidity, now a new feature, the removal of manganese, is introduced that requires careful manipulation. If it is allowed to pass into the service mains and to the taps of consumers, it will cause rust spots on clothing, stains on plumbing fixtures, etc.

The removal of manganese has been accomplished in a number of

ways. At Baltimore (6) they resort to iron and lime coagulation, which at the isoelectric point of pH 9.2 to 9.4, removes it. Aeration (4) with a slightly alkaline water followed by filtration through a bed of fine coke has been found successful. Weston (3) calls attention to the difficulties encountered in aeration of soft waters, low in iron and pH value and high in organic matter. He further states that "others require the addition of an oxidizing agent, like chlorine or permanganate, to reduce the inhibiting organic matter." In water softening processes using the lime-soda method and in the zeolite process, manganese is usually removed as a part of the normal operation of the plant.

For a small plant, such as the one at Clarksburg, W. Va., situated on the West Fork of the Monongahela River, which Boynton has charge of, the solution of the problem has been attained by the addition of lime beyond the amount necessary to produce alkalinity to take care of the required alum. This has been added as a secondary dose to the water just before it reaches the filter beds, maintaining a pH of approximately 8.3, with a residual pH in the filtered water of 7.8 to 8.0. At times when the turbidity in the river water is almost nil, alum is dispensed with, and the lime dosage increased to the aforesaid pH of 8.3, thus removing the manganese in the coagulation basins, and producing a large lime-floc, so-called, which aids filtration as the floc of alum does. These two points of application of the lime represent two phases of the same answer to the problem of manganese removal. The removal of free CO_2 at the same time avoids the resolution of manganese from the deposits on the sand, where it collects as oxidized manganese probably as the hydrated "ic" oxide.

It is probably present in the streams as the carbonates and sulphates in solution, its original source being the mine drains which pollute all the streams in this vicinity. That the manganese remains in solution until the water reaches the filter plant seems to be borne out by the fact that, during flood stage, it is present in less amount than during dry or concentrated periods, whereas the iron from the same source is readily oxidized and reaches the plant in suspension in rapidly increasing amounts. The manganese shows the effect of dilution, being in solution, while the precipitated iron is brought along with the other suspended matter as ferric hydroxide, as high as 10 to 20 parts per million in toto and 0.1 to 0.2 in solution. At times of drought the manganese has occurred as high as 4.0 parts in a clear water in the river with iron very low. This point has been

emphasized to show that these two elements which are considered similar are subject to vastly different reactions, or that the same reactions may occur with manganese as with the iron, but more slowly. At Fairmont, W. Va., on the Tygarts Valley River, the water carried 7.0 parts of manganese, during the drought of 1930. The water of the West Fork is nearly always alkaline, slightly acid for short periods, while that of the Tygarts Valley is nearly always acid, slightly alkaline for short periods.

The effect of manganese upon the physical or mechanical parts of the filter plant is very noticeable. The first indication of the presence of manganese was observed on the bowl of the chlorinator (Wallace and Tiernan suction type) almost simultaneously at Clarksburg and Fairmont, about seven years ago. A brownish deposit accumulated on the bowl of the chlorinator and the water would become turbid when the flow was interrupted for any reason. This was identified as a salt of manganese. The next step was to turn to the filter sand which was covered with a dark brown deposit which when wet gave the sand the appearance of soft coal dust. Previous to this time the deposit on the sand beds had been supposed to be an accumulation of aluminum hydroxide and coagulated mud, although surprise was felt at the color, and that such an amount should collect with good filter washes. The fact that an air-wash has been employed at this plant has probably been responsible for the absence of cracks in the filter beds, as at other plants, where manganese is present in the water, such cracks have occurred and have been eliminated by the introduction of air-wash. Langelier and Decosta (7) treating a water containing 0.1 p.p.m. manganese had difficulty with sand cracking. Two years after the plant started operation the sand coating amounted to 20 percent by weight and there was much underwater cracking. The introduction of air wash into the filters eliminated this cracking and seemed to improve operating conditions.

The manganese incrustation at Clarksburg was found by both the authors independently to be 1.02 to 1.03 percent of total weight of sand and detritus. The effective size of the sand grains had increased from 0.35 to 0.44 to 0.65 mm. and the uniformity coefficient was 1.38, originally 1.65, as analyzed under Professor Carpenter's direction at the West Virginia University, Morgantown. It is a moot question whether before the introduction of the excess lime treatment only part of the manganese was precipitated out upon the sand grains and a part passed through in the filtered water, or whether

all was removed and some dissolved by the passage of water containing free CO_2.

The effect of the precipitated manganese oxide upon the filter runs is hard to state definitely. While the average of filter runs has dropped from 90 in 1925 to 68 hours in 1931, the decrease has not been uniform or wholly progressive and filters are still running 100 hours between washes before attaining a loss of head of 10 feet, the point at which filters are washed. Other things influence the length of runs, of course, such as the growth of micro-organisms in warm weather and the frequency of muddy rivers and heavy alum dosage, although the turbidity on the filters rarely goes above 5 to 10 p.p.m. even with muddiest water. We have been very lenient in washing for visitors, although not making a practice of washing even for them when the loss of head is less than 5 feet.

In washing the filters air at 2.5 pounds pressure is applied through the wash water distribution system for a period of 2 or 3 minutes. The air is shut off and wash water applied with a vertical rise of 9 to 10 inches per minute for a period of 5 to 8 minutes. As the capacity of the plant is greater than the consumption, filters are allowed to stand after washing with no rewash, until such time as they are needed. The average amount of wash water has been 0.6 percent for the last 6 or 7 years.

The bacterial efficiency of the filters has remained about the same for the past six years, 70 to 77 percent, determined by samples taken from applied water and from the effluent of the filters, excluding figures obtained during the drought of 1930, when the growths on the filters vitiated results. The B. coli index of the filtered water has shown a steady decrease during the past seven years, from 4.6 to 1.9 B. coli per 100 cc.

Janzig and Montank (8) found that idle filters after being placed in operation, showed large quantities of manganese for 12 hours. At Clarksburg it is quite common to let filters stand idle for several days, but no similar difficulty of high manganese has been encountered when the filters were placed in operation.

During the late winter all of the filters were renovated and the beds showed staining all the way to strainers and the concrete forming the filter was badly stained. Under-drains showed deposits of iron and manganese and they have been replaced by cement lined cast iron pipe. The sand and fine gravel were removed from the bed by means of water jet and hopper. The sand did not seem to improve

by this washing, but the deposits on the fine gravel washed off quite readily and probably could have been reclaimed by passing it through several sand hoppers.

As this sand was used for filtration of a manganese-bearing water, without attempt to remove manganese for several years, it will prove an interesting study to examine the sand over the next few years to determine the presence of manganese at various depths, as the excess lime treatment will be used continuously. It will also be interesting to see if manganese will pass the bed before the sand becomes coated.

The writers wish to express their appreciation to the State Sanitary Engineers who kindly furnished data on manganese troubles in their various states.

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DISCUSSION

Arthur F. Mellen:[3] The authors of this paper are to be commended for their contribution to the subject of manganese troubles.

Pronounced sand caking and underwater cracking are not observed in our filters. This may be due to the fact that our wash water is applied at a vertical rise of about 18 inches which, while a low velocity wash, is still approximately twice that in use at the Clarksburg plant.

As to the trouble reported at Carbondale, Illinois, it would be interesting to know what could cause the manganese to be dissolved off of the sand following copper sulphate treatment of the reservoir.

Our trouble with re-solution of manganese has come entirely as the result of leaving filters idle over long periods with water standing in them. It has been found that the water near the sand surface is practically devoid of oxygen and this is apparently a factor in this phenomenon. No manganese trouble is encountered if filters are dry when idle.

[3] Filtration Engineer, Water Works Department, Minneapolis, Minn.

Our experience with incrustation due to manganese is very similar to that of Clarksburg, in that the effective size of the original sand and of the present sand is very close to that reported. The increased size of the sand grains should tend to lengthen the filter runs. As the authors point out, many influences affect the filter runs and it is well known that it is very difficult properly to evaluate each of these various factors.

We believe that the presence of manganese on filter sand may assume importance in a number of directions and research work on this subject is being continued in our laboratory by Messrs. Janzig and Montank who have very kindly advised me in preparing this discussion.

ROBERT SPURR WESTON:[4] Manganese, the presence of which was so seldom recognized by the analysts of the last generation, seems to have made its presence increasingly known of late, partly because of its interference with the ortho-tolidine test, partly because of its ready deposition on the alkaline surfaces of concrete lined tunnels, conduits and pipes, and partly because of its ready passage through certain rapid filters and its accumulation in the filter material of others,—the latter the chief subject of the authors' scholarly paper.

The speaker well remembers the filters of certain deferrization plants, notably at Stralsund and other plants in Northern Germany, where the readily coagulated iron formed a reddish deposit at the surface and the more slowly oxidizing manganese a black zone in the lower depths of the filter material with a zone of clean material between.

The deposition of manganese in distribution systems has long been a nuisance and is usually greatest in comparison with the deposition of iron at the ends of the systems.

These depositions have been troublesome in soft water regions where conduits of concrete have been used, notably at New York, Providence and other cities along the Atlantic seaboard supplied with upland water. Should treatment fail at Clarkburg, fear for the cement lined under-drains might be entertained.

The source of manganese in these cases is usually the soil of reservoir bottoms, the solvent the acids produced by the decomposition of organic matter.

In the Wanaque (N. J.) reservoir (29,000 Mg.), the amount of

[4] Of Weston and Sampson, Consulting Engineers, Boston, Mass.

manganese in the water has decreased as the reservoir has aged and the accumulated organic matter has decomposed.

Particularly troublesome is manganese in waters of high hydrogen ion concentration. In these, one must be careful not to raise the pH value too high lest manganese be precipitated.

Excepting during experiments at Pittsburgh twenty years ago, the speaker has had little experience with such highly mineralized and buffered waters, containing manganese, as the authors describe. In these waters the pH value may be high without preventing the

TABLE 5

Effect of pH value upon the removal of manganese from water by coagulation with ferric sulfate ("chlorinated copperas")

Raw water:						
Color (p.p.m.)	25	25	25	25	25	25
Turbidity (p.p.m.)	25	25	25	25	25	25
pH value	7.8	7.8	7.8	7.8	7.8	7.8
Iron (p.p.m.)	0.09	0.09	0.09	0.09	0.09	0.09
Manganese (p.p.m.)	1.0	1.0	1.0	1.0	1.0	1.0
Chlorinated copperas (p.p.m.)	10	10	10	10	10	10
Hydrated lime (p.p.m.)	0	10	20	30	40	50
Period of slow stirring (mins.)	15	15	15	15	15	15
Period of coagulation (hrs.)	24	24	24	24	24	24
Coagulation	Fair	Good	Ex.	Ex.	Ex.	Ex.
Filtered water:						
Color (p.p.m.)	45	7	4	5	5	5
pH value	7.4	7.9	8.8	9.0	9.1	9.2
Iron (p.p.m.)	0.85	0.03	0.00	0.00	0.00	0.00
Manganese (p.p.m.)	0.56	0.35	0.35	0.04	0.00	0.00
Appearance	Fair	Good	Ex.	Ex.	Ex.	Ex.

use of sulfate of alumina as a coagulant. At Baltimore, and more recently, at Providence, sulfate of alumina failed to remove manganese, and iron had to be substituted for aluminum for use with waters raised to a sufficiently high pH value to effect the removal of manganese.

At Clarksburg the authors apparently found no difficulty in coagulating the aluminum hydrate, but one asks if the manganese content is not high enough to act as a coagulant and counteract the tendency of the aluminum hydrate to dissolve in so alkaline a water.

Some experiments made in 1931 with a river water in Pennsylvania illustrate the principles involved. At times this water contains high

amounts of manganese. The deposits in the distribution system, although the water has been treated with lime and sulfate of alumina, as at Clarksburg, contain as high as 7 percent of manganese.

While the more economical treatment of this water was to coagulate it with sulfate of alumina with the pH value held at 6.6, no manganese could be removed at so low pH values. Neither was it possible to remove the manganese by raw water chlorination followed by alum treatment, and the addition of lime with sulfate of alumina was a decided disadvantage.

Either ferrous sulfate, or "chlorinated copperas" with lime readily removed the manganese when the pH value was maintained at 9.1.

Table 5 illustrates the effect of the pH value, and also shows how much more readily iron in surface water is removed than is manganese.

Many other experiments confirmed the figures in table 5 and showed the treatment to be superior to treatment with commercial ferric sulfate and lime.

Where waters contain compounds of manganese and organic matter, raw water chlorination or treatment with other oxidizing agents often is of great benefit.

FILTER PROBLEMS AND WATER SOFTENING

By Charles H. Spaulding[1]

The problems connected with filtering water which has been softened with lime and soda, or with lime alone, are not necessarily more troublesome than those of filtering other treated waters. In some cases they are fewer and simpler than they would be without softening. When treated and settled water is brought to the filters in chemical balance, and with turbidity reduced to only one or two parts per million, it may well be expected that filter runs will be long and filter problems reduced to a minimum. Such is the case. Runs of 100 to 200 hours are not uncommon at some plants. Algae interference with filter runs is but little known. It is true that crustacea such as Cyclops and Daphnia sometimes collect in drifts on filter beds in softening plants and at times insect larvae may reach the filters in great numbers, but the smaller microscopic plankton which shorten filter runs are so thoroughly removed in the settling basins that they are not noticed in the applied water at all.

Judging by the results obtained at times, one would seem to be justified in proclaiming the gospel of perfection in treatment as the cure or preventive of all filter troubles in a water softening plant, and this should indeed be kept in mind. It is along this line that water softening practice has been moving. But this gospel is only another way of saying that filters will work well if they have nothing to do.

The filter end of a water softening plant represents an important item of capital investment and might reasonably be expected to perform in proportion to the cost. When, however, any considerable part of the work of a softening plant falls upon the filters, either through necessity or by the direction of the operator, weaknesses appear which indicate that the design of filters has not been given enough study in its relation to the particular needs of a softening plant. The water softening plant filter is a hand-me-down from the simple coagulating plant.

[1] Chemist and Superintendent, Water Purification, Springfield, Ill.

A result of this circumstance is that most water softening plants do have filter problems which, if not embarrassing, at least deserve study to the end that the filter may be better adapted to its function. Either the filter may be made more useful so as to justify the capital investment and operating cost, or the cost may be reduced, or both results secured in some measure.

CHARACTERISTICS OF LIME-SODA TREATED WATER

There are several respects in which lime-soda treated water is unique. First of all, the pH is high, ranging from 8.0 to 11.0. Even if recarbonation is practiced, the pH is seldom reduced below 8.0, while the more usual figure is around 9.0. If recarbonation is not employed, the pH of the applied water is frequently 9.6 or more.

Next there is but little true floc in the applied water unless coagulant is added after softening and settling, or unless the pH is high enough to sustain magnesium hydroxide. This lack of floc tends to produce long filter runs with moderately coarse sand (effective size of 0.45 mm., for example). On the other hand, it may preclude the use of very coarse sand such as is being indicated by experiments in simple coagulating plants.

Another difference is the presence of unsettled calcium carbonate which has to be removed by filtration.

Finally, the water may contain calcium carbonate and magnesium hydroxide in unstable solution, ready to plate out upon the sand grains or other material in the filter.

FUNCTIONS AND DIFFICULTIES OF THE FILTER

There are several functions, one or all of which the filter may be called upon to perform. First to be mentioned is the usual job of removing bacteria, algae, turbidity and floc. These burdens are ordinarily reduced in a softening plant by the character of the treatment, or, to be specific, by the heavy, quick settling precipitates. However, the same sort of problems are encountered as in coagulating plants. Filters may pass floc or turbidity. Mud balls may form and filter beds shrink and crack. The same questions as to sand size and wash water velocity arise, modified by the character of the floc or its absence in many cases. These problems are being studied seriously in a number of coagulating plants. Similar studies should be made in softening plants wherever possible, in order that the facts which are being learned may not be misapplied, because other problems peculiar to water softening plants are concurrent.

Another purpose of the filter is the removal of precipitated calcium carbonate and magnesium hydroxide which have not settled out. The calcium carbonate is crystalline in character, easily removed by ordinary filters without rapid increase in loss of head, but difficult to dispose of by the ordinary filter washing process. It forms a sludge cake over the filter bed, seemingly soft and friable yet sufficiently coherent that when the filter is back-washed, this cake breaks into pieces, some an inch or more in width, floating up and down through the sand. The abrasive action of the sand breaks off small particles that escape to the wash troughs, but when the washing is finished a considerable portion remains on and in the bed. Being of lighter specific gravity than the sand, it usually stops at or near the surface, but some pieces, on account of size, shape or adherent sand, settle to the surface of the gravel layer. This is one of the first steps toward permanent fouling of the filter bed.

The sludge cake may be so soft after first forming that it cannot be picked up without disintegrating, but if the pieces remain unbroken a calcareous shell gradually forms, giving the appearance of gravel, although the lumps may still have soft centers. These differ from mud balls in that there is little or no sand in them, although sand grains may be attached or imbedded incidentally.

If magnesium hydroxide is present, the cohesion within the lumps is greater and fouling of the bed is more rapid. As the sludge cakes composed of calcium and magnesium settle and work into the gravel, a crust forms over the surface of the gravel in uneven patches analogous to the hard spots in ordinary practice; but unfortunately this crust becomes hard and flinty, so that eventually it may have to be removed with a pick. It does permit the passage of water in spite of its flinty nature, but interferes more or less seriously with the even washing of the filter.

Another function of the filter may be the removal of calcium carbonate and magnesium hydroxide from unstable solution. This is the plating-out process which builds up the sand grains and with which all are familiar. It can be shown easily that this process is not in itself uneconomical. The hardness which can be removed in this way will pay for sand several times over, basing its value on the cost of lime to remove an equivalent amount. One reason for this economy is that the plating-out process takes place upon the sludge which is accumulating during the filter run as well as upon the sand grains themselves. The sand incrustation is at first confined to the

upper section of the bed. The increase in size of grains is balanced by the lower specific gravity of the coating, so that for as much as several years the lower part of the bed may be free from incrusted sand. Gradually, however, as the size increases, the incrusted grains extend to the gravel bed. As scaling proceeds further, the bed requires increasing upward velocities of wash water to float and better preparation of the applied water, to obtain a clear effluent until the limit of practical operation is reached. The sand grains by this time are nearly perfect spheres extremely uniform in size.

One drawback to this process is cementation of the gravel bed previously mentioned, which may go on at the same time. This cementation is probably due in part to the action of water of high pH on siliceous material. Whoever has observed the stoppers stick in bottles of alkali or the freezing together of glass beads in alkaline solution will understand how siliceous gravel grows together under the influence of alkaline water, plus sludge cake from the top of the filter and a small accretion of calcium carbonate from solution. It seems likely that a considerable part of this difficulty could be overcome by the use of more suitable material than siliceous sand and gravel, and by keeping sludge cake at the surface of the bed.

Siliceous sand is more soluble in water of high pH than in low pH. In industrial practice, magnetite, calcite and other materials have been used with hot process softeners to avoid this solubility. Powell[2] cites boiler scale analyses showing 85.8 percent calcium silicate in a case where a sand filter was used on hot softened water as compared to zero silicate where magnetite was used. The sand is not so soluble in cold water, but silicate scale is nevertheless found in some cases where cold, lime-softened water is filtered through sand, and silica is prominent in the acid washings from cemented filter gravel. The solvent action is naturally stopped by the formation of carbonate scale, but by the time this is complete, new sand may be required.

POSSIBLE CONTROLS

These problems may now be correlated with the pH of the applied water. Case No. 1 includes water which has been stabilized with CO_2 to a pH of 8.4 or less. The problems in this case are of the same general character as in simple coagulating plants modified by efficient clarification. Case No. 2 A includes the range of pH from 8.6 to 9.6

[2] "Boiler Feed Water Purification." McGraw-Hill Co., 1927.

in which caustic alkalinity is not present. This represents water which has been either slightly undertreated or, if overtreated, recarbonated, but not to complete stability. Such water will plate out calcium carbonate on the sludge and on the sand grains. The sludge cake will be sufficiently cohesive to resist complete removal in the ordinary washing process and will eventually foul the filter unless special prevention is effective. Case No. 2 B includes the same range of pH as No. 2 A, but applies to water which has received the excess lime treatment followed by recarbonation and stabilization with carbonate sludge to produce a pH about 9.3, as a rule. Such water will deposit sludge cake in the filters with conventional washing procedure, but little or no scaling of sand occurs. No cementing of gravel has been observed so far, but the process is only about two years old.

Case No. 3 includes the range from 9.6 to 10.6, or more. Such water, if it contains calcium and magnesium, will deposit the most adherent and troublesome sludge cake. The water will scale sand grains rapidly and cement the gravel as well as the sand itself.

Recent experience with experimental glass filters indicate that the cure for filter washing difficulties in a water softening plant should be sought in some variety of surface wash. With any grade of sand or any velocity of vertical rise of wash water or sand expansion within practical limits, broken sludge cake from the surface can be seen to rise and fall through the bed; whereas the floc which penetrated the surface during the filter run is almost instantly released when the flow of water is reversed.

The surface wash described by Baylis, and consisting of a grid of perforated pipe suspended above the sand bed, has been used at one plant employing excess lime treatment followed by recarbonation. Washing has materially improved. Mud balls have been eliminated and the sand grains kept clean. The sludge cake remains, however, although cleaned and polished by the washing process.

Disintegration of the cake before the vertical wash begins is indicated as a means of preventing migration into the bed during washing, and this has been attempted by applying a spray to the surface before the wash, first draining the water below the surface. At a plant where 75 pounds water pressure is available, a spray nozzle attached to a $\frac{3}{4}$-inch hose has been used with promising results, although there are practical obstacles in the way of getting perfect results by this method. The experience in this case shows an apparent saving of

25 percent or more of wash water as well as material reduction in the amount of cake residue left on the filter.

Just how serious the problems which have been mentioned may become depends on too many local circumstances to attempt an average. It is believed that water of pH less than 8.4 may extend the life of a filter indefinitely, depending on other factors than the softening action itself.

In the next higher zone of pH up to 9.6, it may also be that long service will be obtained, but the present washing methods require improving, or else sand may require rescreening or special washing probably once in five years. In addition, the gravel layer will require close watching to prevent formation of cemented spots which can be broken up by crushing if attacked early.

In the top zone of pH above 9.6, a life of ten years or less for the bed may be the limit. It seems likely that more suitable material and adequate surface washing might make this zone practical from a filtration standpoint, but under present conditions the cementing and fouling problems will make maintenance a troublesome problem in a large plant.

This discussion has been confined largely to the problems of filter washing. The questions of optimum sand size and material have barely been mentioned to point out that they are not necessarily the same as for coagulating plants. Solutions of these problems appear to lie in a concerted study by filter operators along the lines of that promoted by the committee on filtering materials of the American Society of Civil Engineers.

FURTHER EXPERIENCE WITH HIGH RATE OF FILTER WASH

By W. C. Lawrence[1]

In the fall of 1925, the Baldwin Filtration Plant was placed in service, and for the three succeeding years, the filters were operated at the normal rate of wash of 24 inches rise per minute. After these three years of operation, the sand beds assumed the condition that prevails in most plants that have been in service any length of time. Cracks appeared, especially around the edges of the filter beds, small mud balls had begun to form, and a piling up of sand and organic matter against the walls of the filter boxes took place.

After conducting some laboratory experiments with glass tube filters using Mr. Hazen's formula for sand expansion and using one of the large filters to verify the laboratory work, I then started on January 1, 1929, using a 50 percent sand expansion in washing filters on all but four of the large units which were kept for control purposes. The schedule used in raising the wash water stems for a 50 percent expansion was stated in the previous paper on this subject, that is, increasing the rise approximately 4 inches per minute for every ten degrees Fahrenheit starting at a 24 inch rise at 32°F. up to a 39 inch rise for 72°F. and up, and then back down to 24 inch rise from 42°F. to 32°F.

At the highest rate of wash, we lost some fine sand due to the sand grains being coated with organic matter, thereby having a higher bouyancy, so at the present writing the maximum rate of wash in the summer time does not exceed a 37½ inch rise per minute.

The results for the first year, 1929, were very gratifying although some sand was lost. In 1930, we washed all beds at a 40 per cent sand expansion; this year's results were not so satisfactory.

In 1931, the 1929 program was carried out with some slight changes in adjusting the valve stems, but not deviating from the 50 percent sand expansion. This present year we have adopted, for the same expansion, a finer adjustment of the rises of wash water, changing

[1] Superintendent of Filtration, Baldwin Filtration Plant, Cleveland, Ohio.

with closer variations in temperatures. This requires the operators to make changes as often as once a week when the water is warming up rapidly.

Another routine operation in connection with this work is taking out 25 percent of the filters every two weeks throughout the year and draining them through the filter waste and then raking them. This dries up any organic matter that might form and is immediately washed off after the first wash before placing these filters back in service.

From the above work, a comparison of the following sand analyses is shown in table 1:

(a) Sand prior to placing—as shipped to the Baldwin Filtration plant in August and September, 1925.

(b) The top 6 inches of the filter beds nine months after operation with normal filter wash.

(c) A composite of all filters at the end of the year, 1926.

(d) After five years operation—including 1929 with 50 percent expansion and 1930 with 40 percent expansion.

(e) A composite of the forty filters for six years including the year, 1931.

TABLE 1

	EFFECTIVE SIZE (MM.)			SIZE FINER THAN 60 percent (MM.)			UNIFORMITY COEFFICIENT		
	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum
(a)	0.385	0.455	0.340	0.617	0.725	0.560	1.600	1.750	1.365
(b)	0.366	0.400	0.315	0.444	0.495	0.400	1.215	1.297	1.150
(c)	0.397			0.488			1.230		
(d)	0.409	0.425	0.375	0.498	0.555	0.470	1.212	1.297	1.109
(e)	0.400			0.490			1.225		

From a comparison of the data in table 1, the effective size of sand is not much different after six years from the original sand. The finer sand than 60 percent has been reduced from an average of 0.617 to 0.490, and the uniformity coefficient from an average of 1.600 to 1.225.

The depth of the sand bed was originally 30 inches which has been reduced to approximately 27 inches. Part of this three inch difference was lost at the beginning of operation of the plant, but most of

it when we began washing with a 50 per cent expansion during the year, 1929, when the sand was heavily coated and at its worst condition. In the past year we have lost practically no sand. We still have a 27 inch bed which is more than ample for good filtration.

TABLE 2

YEAR	BACTERIA PER 20°C.		BACTERIA PER 37°C.		B. COLI INDEX PER 100 cc.	
	Raw	Filter effluent	Raw	Filter effluent	Raw	Filter effluent
1926	1,172	252	320	61	576.8	13.6
1927	1,912	484	594	73	219.0	11.3
1928	605	113	238	42	118.9	6.8
Percent reduction...	77.0		84.7		96.5	
1929	646	86	242	25	383.5	23.9
1930	332	73	167	43	659.7	26.9
1931	665	85	487	46	961.1	19.7
Percent reduction...	85.1		87.3		96.5	

TABLE 3

YEAR	WASH WATER USED (M.G.)	AVERAGE PERIOD OF SERVICE (HOURS)	RATE OF RISE (INCHES)	PER CENT OF WASH WATER USED
1926	956.4	11	24	2.72
1927	767.1	17	24	2.46
1928	583.1	24	24	1.93
Average.....	768.9	16	24	2.32
1929	546.9	29	29.8	1.72
1930	517.8	25	29.3	1.57
1931	470.4	29	30.6	1.65
Average.....	511.7	27	29.9	1.61

The sand at the present time is in good condition, free from organic matter and of a color the same as the original sand.

For taste and bacterial purposes the writer has adopted pre-ammoniation and pre-chlorination the first of April, so the sand should remain in its good condition at all times.

From the bacterial results very little differences are shown. The six years of plant operation are shown in table 2. The first three

had a 24 inch rise of wash water, and the last three a 50, 40 and 50 percent sand expansion, respectively.

This may possibly be due to the organic matter that had accumulated being scoured off and releasing bacteria, or acting as food for propagation.

Quite a difference is noted from the tabulation of the amount of wash water used and the length of filter runs, as shown in table 3.

As shown in table 3 the total yearly amount of wash water used over two periods of three years each, was reduced 257 million gallons a year or a reduction of approximately one-third, and an increase of 69 per cent in the hourly period of service for each filter.

The total amount of wash water saved over the three year periods is 771.5 million gallons and, at our cost of $40 per million gallons, totals $30,860.

From all the above tabulations, I feel that consulting and designing engineers should construct filters with larger troughs, larger gutters and sewer valves, adequate strainer systems, and ample heads in wash water tanks to obtain these higher wash water rates.

DISCUSSION

R. W. FURMAN:[2] In connection with the discussion of filter washing it appears that not enough importance is attached to two details, namely, the necessity of thoroughly cleaning the sand at each washing operation, and secondly, the effect of changes of floc on the filter sand through disintegration between wash periods.

It is assumed that the various filters are correctly designed and that the correct size of sand has been selected for the particular water to be treated. At the Toledo plant a sand having an effective size of 0.38 mm. is required in order to secure a floc retention on the extreme top surface of the sand as desired.

For the purpose of comparison, two types of floc are designated, primary floc and secondary floc. Primary floc is one that is newly formed and has not undergone any of the changes due to decomposition. This type is most easily retained on the top of the sand, is thoroughly removed by the washing operation if sufficient wash water is used, and does not have the tendency to break up into smaller particles with subsequent penetration of the lower sand layers during high filtration rates. If the treatment is so adjusted that a floc of

[2] Superintendent of Filtration, Toledo, O.

this type is produced, it has been found that by preventing floc disintegration, after the deposit on the filter sand, a complete removal may be effected without an excessive increase of wash water.

Secondary floc is described as one that has undergone decomposition in the sedimentation basin or one that, due to too long a period between filter washings, has started to break down in the filter itself. In either case it has been noted that the same results may be expected. Small fragments, undergoing decomposition, are drawn into the filter sand, and, due to their gelatinous nature, are able to attach themselves rigidly to the sand particles becoming extremely difficult to remove. Such a condition, in many cases, is immediately shown by excessive sand penetration, although this may also be caused by improper treatment or an insufficient application of wash water.

Should such conditions occur, they may be corrected, in most cases, by resorting to a double wash. By double washing is meant two or more operations as follows, which have in practically every instance shown immediate improvement. If a filter shows indications of the appearance of secondary floc, or if sand penetration is indicated, a wash of approximately 3 minutes is used, with the maximum amount of wash water allowable without sand loss, this to be based on the water temperature and the sand expansion. Wash water is turned off at the end of 3 minutes regardless of whether the sand is clean or not, and the sand is allowed to grade hydraulically until it becomes compact. This operation is of importance inasmuch as the second breaking of the filter has the effect of causing sufficient erosion, by the rubbing of sand particles against each other, to dislodge much of the adhering deposit that is not removed while the sand is in an aqueous suspension of one single washing operation. The second wash does not as a rule require the same length of time as the first, 1 to 2 minutes frequently being sufficient. A third wash or even more drastic remedies may be needed if mud balls and clogging are excessive.

At the Toledo plant which has 56 filters of 1 m.g.d. each and having a flashy supply to treat that always has a heavy organic load, much difficulty was experienced from 1910–1916 in the care and operation of the filters. Cracking, shrinking, and clogging followed by the formation of mud balls seriously influenced the quality and quantity of the water produced. Several years of experiments followed from 1916 and ultimately the foregoing procedure was es-

tablished. Since 1919 no difficulties have been encountered that could not be corrected by the use of proper wash methods.

Although the Toledo plant is equipped with air agitation in connection with the filter washing, it has been demonstrated, in 9 other plants that had high velocity wash only, that the same washing processes were sufficient to prevent their filter sand troubles if properly applied. In more recent years the care of filters at Toledo have become a matter of major importance. Algae growth has reduced the filter runs to as low as 2 hour periods and the presence of excessive amounts of crustaceans has necessitated extreme care in the operation of all stages of the process. For the past two years it has been necessary to operate the filters at rates from 20 to 30 percent in excess of their rated capacities in the months of June, July, and August. Here, at least, the importance of satisfactory filter operation cannot be over-stressed.

The average amount of wash water used at this plant over a 10 year period is 2.58 percent in comparison with a previous 10 year average of 3.92 percent and clogged filters. At the same time the cost of chemical treatment has been materially reduced. Should the filter wash be insufficient, broken floc is partially or completely distributed in the sand when the wash water is turned off and the length of time that elapses before this material passes into the effluent is largely dependent on how far this penetration has gone, the rate of filtration, etc. It is essential that a filter be thoroughly cleaned at every washing operation and that too long a period between washes does not create conditions that are hard to correct.

WATER SOFTENING BY ZEOLITE IN MUNICIPALITIES

By S. B. Applebaum[1]

Interest in the softening of municipal water supplies has been steadily increasing during the last few years. C. P. Hoover (1) in 1930 stated that in 1929 there were approximately 110 cities and villages in the United States that were softening their municipal supplies. The total number of cities in the country having a population of 2500 or over is in the vicinity of 3000 and it is safe to say that over 50 per cent of the total population of the country resides in these cities of 2500 population and over. Of the 110 cities above mentioned that are softening their municipal supplies, about 30 per cent are in the State of Ohio. We have therefore taken the State of Ohio in a further study of this matter and figure 1 shows the growth in number of installations in municipalities in Ohio from 1903 to 1930. The upper curve gives the number of installations each year and the lower curve gives the soft water populations that have been added each year. This curve shows that the soft water population is increasing at present at the rate of about 25,000 per annum.

VALUE OF SOFT WATER TO A COMMUNITY

This growth of soft water population has resulted from a growing appreciation on the part of the community of the value of soft water. The advantages of soft water have been thoroughly described in technical literature on the subject and therefore only a summary need be given here.

Destruction of soap

Hard waters are hard to wash in because they destroy soap, forming insoluble, sticky curds which tend to adhere to everything with which they come in contact. A large excess of soap must be used before a lather can be formed. Hard waters, therefore, waste soap and make thorough cleansing difficult. The insoluble curds formed

[1] Vice-President, The Permutit Company, New York, N. Y.

stick to garments, to the sides of washbowls and bathtubs, to the hair and to the skin.

Effect of hard water on linens

Garments laundered in hard water have a harsh feel, a yellowish or grayish cast, and a rancid odor, which are due to the insoluble curds adhering to them. When the garments are ironed this adhering deposit stiffens and discolors the materials. Experience indicates that continual laundering in hard water decreases the life of garments

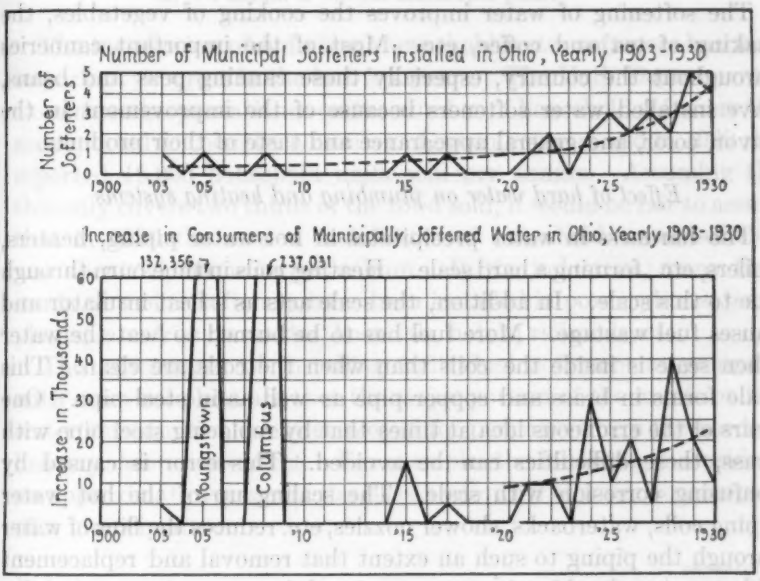

FIG. 1. GROWTH OF MUNICIPAL WATER SOFTENERS IN OHIO

from 20 to 40 percent. The use of chemicals, such as lye, washing soda, ammonia, etc. decreases but does not eliminate these troubles. Furthermore, the excesses of such chemicals used may have a harmful effect on the garments laundered, particularly on silks and woolens.

Effect of hard water on porcelain

Insoluble curds stick to the side of porcelain washbowls and bathtubs, attract soil and grime to them and make the usual ring which can only be removed by scouring. It is usually necessary to employ

a scouring powder or similar material and the use of such abrasives in time dulls and mars the glossy finish of porcelain.

Effect of hard water in shampooing

The adherence of the insoluble curds to the skin and hair interferes with shampooing. The large number of individual water softeners installed in beauty parlors throughout the country testifies to the importance of soft water for this purpose.

Effect of hard water in cooking

The softening of water improves the cooking of vegetables, the making of tea and coffee, etc. Most of the important canneries throughout the country, especially those canning peas and beans, have installed water softeners because of the improvement in the flavor, color, and general appearance and taste of their products.

Effect of hard water on plumbing and heating systems

The hardness in water precipitates in hot water piping, heaters, boilers, etc., forming a hard scale. Heating coils in time burn through due to this scale. In addition, the scale acts as a heat insulator and causes fuel wastage. More fuel has to be burned to heat the water when scale is inside the coils than when the coils are clean. This scale forms in brass and copper pipe as well as in steel pipe. One hears of the erroneous idea at times that by replacing steel pipe with brass, these difficulties can be avoided. This error is caused by confusing corrosion with scale. The scaling up of the hot water piping coils, waterbacks, shower nozzles, etc. reduces the flow of water through the piping to such an extent that removal and replacement of hot water plumbing is necessary, involving expensive repair bills, especially when this piping is concealed in the walls of a house.

The following is an approximate evaluation, for an average family of five people, of the cost of the discomforts caused by the use of hard water, frequently referred to as the "hard water tax" which the family has to pay indirectly:

	dollars per year
Cost of soap, soap powder, washing powder, etc. destroyed by the hardness of the water	16.50
Plumbing bills	6.00
Wear on cooking utensils	2.00
Fuel wastage due to scale	6.00
Decreased life of linens, etc	13.50
	44.00

A "hard water tax" of $40 per family per annum is therefore a conservative statement of the cost of using hard water for household purposes. The industries pay a "hard water tax" that is often far greater per unit volume of water consumed, because the bulk of the water may be used in processes where the hardness is very detrimental.

INDIVIDUAL SOFTENERS VERSUS CENTRAL MUNICIPAL PLANTS

There are two ways of solving the hard water problem:

(1) To let the individual users soften their water
(2) To install a central municipal plant

The installation of individual household water softeners has made important progress in the last few years. The U. S. Department of Commerce keeps statistics on the number of household softeners that are reported to them by a number of manufacturers and in 1929 they reported 18,000 household units sold per annum. Assuming that this only covers two thirds of the total sold, it would be fair to assume that in 1929 the rate of installation of household water softeners was about 25,000 units per annum. In the industries, many thousands of zeolite softeners have been installed in power plants, textile plants, laundries, canneries, hospitals, hotels, railroads, etc.

The advantages of installing individual softeners as compared with a central plant are that the major portion of the municipality's water consumption is not employed for purposes where the hardness has any deleterious effect. For example, the following is considered an approximate average of the classification (4) of water consumption of a municipality:

	per cent of total consumption
For household purposes	35
For industrial purposes	30
For public use, such as street sprinkling, sewer flushing, flushing of city mains, fire extinguishing, etc	10
Loss by leakage from mains, waste to prevent freezing, wilfull or careless waste	25
	100

The household and industrial consumption constitutes about 65 percent of the total and the remainder is used for purposes where the quality of the water is immaterial. Of the 65 percent that is used for household and industrial purposes, a considerable part is consumed

for purposes where the hardness does not matter. For example, in the average home a great deal of the water in summer is used in sprinkling lawns and other summer uses. When an individual water softener is installed, provision can be made by suitable piping connections, to use hard water for the sprinkling of lawns, flushing of toilets, etc. and to use soft water for the remainder. Therefore, the installation of individual water softeners has the advantage over the central municipal softener of greater directness and economy in supplying soft water for uses where the quality of the water is material.

On the other hand, the cost of a central municipal plant, both from the investment and operating standpoint, will be much less than the cost of individual water softeners installed in every home and factory.

In actual practice, where a municipal softener is installed, a compromise is generally reached. If a municipality can afford it, it softens all of the water to a certain degree, but not completely. The individual is then free to finish the softening by installing a softener in the home or factory.

TYPES OF MUNICIPAL SOFTENERS INSTALLED

After a municipality has decided to make an installation of a central water softener, it has the option of two types of water softeners:

(a) Lime soda
(b) Zeolite

The decision as to which of these types is best suited to their conditions can only be made after a thorough study of:

(1) The composition of the water
(2) The variability of composition of the water
(3) The size of the plant
(4) The type of labor that will be available
(5) The engineering conditions surrounding the installation

Several municipalities in Ohio that have decided to install central water softeners are passing through the process of making their choice between lime soda and zeolite at the present moment and a full consideration of this subject is therefore of timely interest.

GROWTH OF MUNICIPAL ZEOLITE WATER SOFTENERS

Table 1 gives a partial list of the municipal zeolite plants installed to date. A careful search has been made for all references to zeolite

TABLE 1

Partial list of municipal zeolite water softeners

PLACE	POPULATION	SOURCE OF SUPPLY	HARDNESS AS $CaCO_3$	TYPE PLANT	INSTALLED
			p.p.m.		
Illinois:					
Mascoutah	2,343	Wells	750	L—S—Z	1921
Zion	3,000	Wells	340	Z	1928
Indiana:					
Edwardsport	750	Wells	686	Z	1930
Iowa:					
Mt. Vernon	1,499	Wells	272	Z	1931
Louisiana:					
Newellton	627	Wells	322	Z	1932
Michigan:					
Clinton	1,026			Z	
Coopersville	1,004	Wells	800	Z	1925
Minnesota:					
Marshall	3,100	Wells	475	Z	1925
New Mexico:					
Gallup	6,000	Wells	433	Z	1930
New Jersey:					
South Orange	7,274	Wells	193	Z	1929
New York:					
Cuylerville	265	Surface	513	Z	1924
Flatbush		Wells	180	Z	1929
Ohio:					
Arcanum	1,149	Wells	386	Z	1930
Burton	597	Wells	129	Z	1930
Findlay	19,363	Surface and wells	500	L—Z	1931
Lowellville	2,550	Wells	285	Z	
Mechanicsburg	1,470	Wells	336	Z	1932
New Bremen	2,000	Wells	514	Z	1930
Wapakoneta	5,378	Wells	380	Z	1930
Cedarville	1,028	Wells	382	Z	1931
Oklahoma:					
Mangum	3,405	Wells	510	Z	1931
Pennsylvania:					
McKees Rock	18,077	Wells	200	Z	1925
Oakdale	1,611	Wells	416	Z	1930
Sewickley	4,955	Surface	265	Z	1929
Springdale	2,929	Wells	340	Z	1929
Woodville	510	Wells	350	Z	
Texas:					
Eastland	9,368	Wells	315	Z	1928
Virginia:					
Dublin	400	Wells	270	Z	1929

L = lime; S = soda; Z = zeolite.

plants, but there are undoubtedly a number that have not been described in the literature and it is for that reason that it is called a partial list. The bulk of these zeolite installations have been made, as the list shows, in the last three or four years, testifying to the present trend towards zeolite. In illustration of some of these zeolite plants that have been recently installed, the following cases are cited:

(*a*) *Sewickley, Pa.* The entire plant has been well described in Campbell's and Davis' paper (2) before the Association in 1929.

The source of supply is an infiltration system under the bed of the Ohio River and at very rare intervals there is a break in this system and turbid river water may enter it. Provision was therefore made in the design of the plant that, when this happened, coagulant may be fed and the zeolite softeners used as sand filters during the emergency. This has actually happened once since the plant was installed and for several days this emergency filtration by zeolite was carried out successfully without damage to the zeolite.

(*b*) *Wapakoneta, Ohio.* Water is pumped from the wells directly to the suction of the high service pumps which discharge the water under pressure through the zeolite softeners to service. The raw water cistern floats on the line between two pumps. In this way a minimum of air is introduced to the water because most of the time the water by-passes the cistern. A pressure solution soda ash feed was installed for the purpose of neutralizing the free CO_2 and permitting a reaction between the soda ash and the hardness present in the effluent (by bypassing a portion of the hard water around the softeners). This was intended to form a protective film on the interior of the piping of the distribution system.

Actually this feed has not been used because no corrosion has been experienced in the distribution system.

(*c*) *New Bremen, Ohio.* The zeolite plant installed at New Bremen, Ohio, successfully removes iron and hardness simultaneously.

CONSIDERATIONS IN FAVOR OF ZEOLITE

Considerations that are leading to a more general adoption of zeolite for water softening are:

Absence of sludge disposal

The water softening reactions accomplished by the zeolite process take place between the soluble calcium and magnesium salts in the

water and the insoluble zeolite by direct contact, without the formation of any precipitates of calcium or magnesium in the water. The effluent of the zeolite softeners is just as clear as the influent and no sludge is produced. The problem of sludge disposal, in connection with a number of existing lime soda plants, is a very serious one and was best described in the Ninth Annual Report of the Ohio Conference on Water Purification, October, 1929, Page 27, as follows:

"The Marion water supply is obtained from deep wells. The water has a hardness of over 700 p.p.m. About two years ago a water softening plant was constructed employing these features: Lime, soda, alum treatment mixing tanks, Dorr clarifiers followed by settling basin and recarbonation. On account of the large amounts of chemicals required for softening more than one million gallons of water a day, the sludge which is formed presents a real problem. There is being produced about 4200 pounds of insoluble material daily. The sludge from the Dorr clarifier contains about 75 to 80 percent moisture. It was first run into an old abandoned reservoir until the reservoir filled up. Nearby the plant was an old barrow plant. This excavation was 400 feet by 75 feet by 20 feet deep. Just a year ago a sludge line was laid to this barrow pit and today the pit is full. . . . . "

"Mr. Cameron, superintendent of filtration of the Fostoria water purification and softening plant, described their problem. The Fostoria plant was designed for lime soda softening. A Dorr clarifier is employed for sludge removal. Softening has not been practiced except for a few days since the plant was put in service, largely on account of the difficulty of disposing of the sludge.

"The flow of the stream into which the plant sewer discharges is very small and in summer there is no flow at all below the plant. If sludge is discharged into this stream, it piles up in one place. When a freshet comes along, it is washed out and deposited all over the land at points below.

"It is now planned to discharge the sludge into an abandoned slow sand filter. When this fills up, an attempt will be made to pump it into the creek when there is a sufficient flow to carry it away, but when the water is not high enough to overflow bottom land and so leave a white sludge deposit. . . . . "

"Mr. Hoover, chemist at the Columbus water works, said that until the rains about a week preceding this meeting, the sludge below the Columbus water works had almost choked up the Scioto River. Speaking of the use of lime sludge for agricultural purposes, he said that unfortunately the sludge is generally produced where the soil does not need neutralization. To use it as a base for fertilizer would upset the necessary combinations in most fertilizers.

. . . .

"Mr. Kimberly, consulting engineer of Columbus, summarized the problem as follows: The increased development of municipal water softening plants has brought to the front the question of water softening sludge disposal. Sludge disposal has been a serious problem in sewage disposal plants for years

and now it appears in the water softening field. It is certainly desirable to find, if possible, some method of properly disposing of sludge. The suggestions which have been made relative to drying the sludge would be all rather expensive and would perhaps unduly increase the cost of operation. It does not seem economically sound to endeavor to recover material from it.

"Current methods of disposal seem to be as follows:

1. Discharge into a large stream either intermittently or continuously.
2. Discharging into small streams during periods of high water. When continuous withdrawal is practiced, storage of sludge is then necessary.
3. Disposal in barrow pits and low grounds.
4. Discharge into city sewers.

"We have no remedies to offer. At the present time there appears to be no good method of taking care of this sludge."

Cost of zeolite plants

As a rule, zeolite plants are less expensive to install because of the simplicity of the equipment as compared with lime soda plants. Taking the average of a number of public bids on zeolite equipment for municipalities in the last few years, a very approximate figure for the cost of a zeolite plant installed, including suitable building, connections to and from the existing piping system, connections to the sewers and any necessary pumps required for the operation of the equipment is $30,000 to $40,000 per million gallons per day, in plants of medium size handling a hardness of approximately 20 to 25 grains per gallon and producing an effluent of 4 or 5 grains per gallon for cities having a population of 2000 to 20,000.

To obtain an idea of the relative cost of lime soda plants, one must consider the type of equipment that is installed. The cost depends to a great extent on the type and detention period of settling tanks employed, the type, size and number of filters used, whether carbonation is included, whether continuous sludge disposal in so-called clarifier equipment is installed, etc.

The most modern types of lime soda treatment plants usually involve the following:

(1) Mixing chambers with suitable motor-driven paddle agitators having a detention period of 30 to 60 minutes.

(2) Settling tanks with proper equipment for sludge disposal, having a detention period of 4 to 6 hours.

(3) Carbonation chamber having a detention period of 15 to 30 minutes.

(4) Filters at a rate of 2 gallons per square foot per minute.

(5) Clearwells, wash water tanks, pumps, chemical feed devices, carbonation equipment, etc.

Recent costs of municipal lime soda plants average about $40,000 to $50,000 per million gallons daily.

TABLE 2

Comparative cost estimates for two million gallon per day softening plant for Sewickley, Pa.*

LIME-SODA		ZEOLITE	
First cost (softening plant only)	$88,000	First cost (softening plant only)	$65,000
Fixed charges	7,040	Fixed charges	5,200
Labor	3,600	Labor	3,600
Maintenance, etc	600	Maintenance, etc	600
Lime, 122 tons at $14	1,708	Salt, 550 tons at $6.50	3,575
Soda-ash, 122 tons at $34	4,148	Additional wash water	105
Alum, 20 tons at $25	500		$13,080
Coke, 55 tons at $12	660		
	$18,256		
Average cost per million gallons daily:		Average cost per million gallons daily:	
Operating	$30.80	Operating	$21.65
Fixed	19.20	Fixed	14.15
Total	$50.00	Total	$35.80

* But pumping only 12 hours per day using 1,000,000 g.p.d.

Cost of operation

Zeolite operating costs are frequently lower than lime soda. The following cases are cited:

In the case of the city of Sewickley, Pa., table 2, taken from the article (2) by J. T. Campbell and D. E. Davis of the J. N. Chester Engineers, Pittsburgh, gives a comparison of a lime soda and zeolite plant. Table 3 gives a similar comparison for a project now under consideration in a town in Kentucky for a capacity of 270,000 gallons in 9 hours. It is to be noted that in the last mentioned comparison the first costs show the lime soda slightly less than zeolite, but this is because the design of this lime soda plant involved steel settling tanks, a very small carbonation chamber consisting of a downtake

in this settling tank, and pressure filters. No mixing chamber was present and the usual meters, controllers, rate of flow and loss of head gauges, etc. which are customary in modern municipal lime soda plants, were not included. The zeolite softener costs, however, included a concrete automatic gravity softening plant of the most modern type.

TABLE 3

Comparative cost estimates for 272,000 gallon per day municipal softening plant on well water

(Analysis below)

	STEEL TANK LIME SODA	AUTOMATIC CONCRETE ZEOLITE
Fixed cost (softening plant and building)	$31,000.00	$33,000.00
Fixed charges	3,840.00	4,100.00
Labor	1,800.00	None
Chemist	2,000.00	None
Maintenance, etc	1,000.00	600.00
Lime—164 tons at $10	1,640.00	
Soda ash—109 tons at $30	3,027.00	4,410.00*
Alum—5 tons at $30	150.00	
Coke—8 tons at $10	80.00	
Total cost per year	$13,737.00	$9,110.00
Average cost per million gallons:		
Operating	$99.00	$50.10
Fixed	38.40	41.00
Total	$137.40	$91.10

Analysis of raw water

Hardness	27.8 gr./gal. as $CaCO_3$
Alkalinity	16.0 gr./gal. as $CaCO_3$
Magnesium	6.3 gr./gal. as $CaCO_3$

*Salt—596 tons at $7.40.

No excess of reagents possible in zeolite-treated water

The zeolite process does not involve the addition of any chemicals. The water passes through a bed of zeolite and by contact the reactions are accomplished without the addition of any reagents. After a certain number of gallons have been softened, the hardness of the effluent begins to rise and at that point the zeolite must be regenerated

by the addition of sodium chloride brine. This brine is in contact with the zeolite for a short time and is then washed to the sewer as a clear liquid containing a mixture of calcium, magnesium and sodium chlorides. The effluent of a zeolite softener contains no excess of any reagent. It is free from carbonates and hydrates and is non-caustic. It contains sodium salts that correspond in amount to that of the calcium and magnesium salts present in the raw water. Since the alkalinity of natural water supplies is bicarbonate in form, the alkalinity of the zeolite softened water is also bicarbonate.

With a lime and soda plant, an excess of lime and an excess of soda ash is required to obtain an efficient softening, especially in the cold, and these excesses of reagents are present in the softened water, unless they are properly neutralized by some type of acid or acid salt. It is for this reason that carbonation has been adopted by many plants recently. Carbonation consists of the addition of carbonic acid into the water for the neutralization of the excess causticity and phenolphthalein alkalinity present.

A recent sample of water received from a municipal lime soda treatment plant in a Southern state, in which no carbonation was employed showed a hardness of 145 p.p.m., methyl alkalinity of 120 p.p.m., and hydrate causticity of 98 p.p.m. It seems almost unbelievable that so high a causticity would be tolerated in a municipal supply, but this water was actually taken out of a tap in the city. The cause of this condition was the absence of a chemist, so that too much of an excess of lime was being added. The hardness was also greater than it should be, due to an insufficiency of soda ash.

Zeolite functions successfully on water supplies that vary in composition. If a water supply increases in hardness, the only effect on a zeolite water softener is that it must be regenerated more often. No other change must be made in the operation of the water softener, nor in the amount of salt employed in its regeneration, nor is the quality of the effluent affected.

With a lime soda plant, as the analysis just given illustrates, a decrease in the hardness of the raw water may cause a dangerous causticity in the effluent unless the amounts of chemicals are reduced and kept adjusted to the varying composition of the raw water.

Absence of after-reactions

Inasmuch as the zeolite process changes the bicarbonates of calcium and magnesium into sodium bicarbonate by base exchange, no car-

bonates or hydrates are present in the zeolite effluent. The water is therefore neutral and no after precipitates are possible.

With a lime soda plant, however, unless the water is properly neutralized by carbonation or an acid salt, the presence of phenolphthalein alkalinity or causticity causes after-reactions which deposit calcium carbonate principally on the filter beds and in the city piping distribution system, including meters, etc. It was due to the troubles caused by these after-reactions that the process of carbonation was adopted in a number of plants. Carbonation, however, is not a simple process to handle. A number of factors must be controlled:

(a) The amount of flue gas pumped must be in proportion to the amount of water to be carbonated.

(b) The composition of the lime pretreated water before carbonation must be constant in its hydrate and carbonate content, or the amount of flue gas must be adjusted accordingly.

(c) The composition of the flue gas must be constant in its CO_2 content, or the amount of flue gas must be adjusted accordingly.

Since the production of the flue gas is usually accomplished by combustion in a small furnace furnished with the carbonation equipment, it is difficult to produce flue gas of a constant CO_2 content. This is hard to obtain even from the furnaces of large boilers with skilled boiler operators in attendance. For these reasons, carbonation is a process that requires considerable skill in handling. If the water is under-carbonated, after-reactions will result in the city mains. If the water is over-carbonated, excess of free CO_2 in the water may depress the pH value. With large plants, as in Columbus, Ohio, where skilled operators are available, there is no doubt that carbonation may be made to function successfully, but with a smaller community the same results are difficult to attain.

No chemical handling

The modern type of lime soda plant involves elaborate mechanical equipment for unloading chemicals from cars to chemical storage houses and involves automatic weighing devices, dust collecting systems, chemical feeding devices, etc., all of which are expensive and complicated mechanical equipment.

The only chemical involved in the operation of zeolite softeners is the common sodium chloride used in its regeneration. The salt is unloaded directly from the boxcar into a concrete basin that

usually holds over a carload of salt stored under water. No dust is involved. In this wet storage, a saturated strength of brine is obtained which facilitates the measuring of the brine for the regeneration of the zeolite. Since the saturated brine is always of practically constant strength it is not necessary to weigh the salt, but merely to measure the brine volume, which gives sufficient accuracy of salt measurement for regeneration. At the right of the wet salt storage basin in figure 2 is shown the brine measuring compartment and a suitable float and scale permits the operator to pump out a definite volume of brine for the regeneration of each unit. The gravel in the

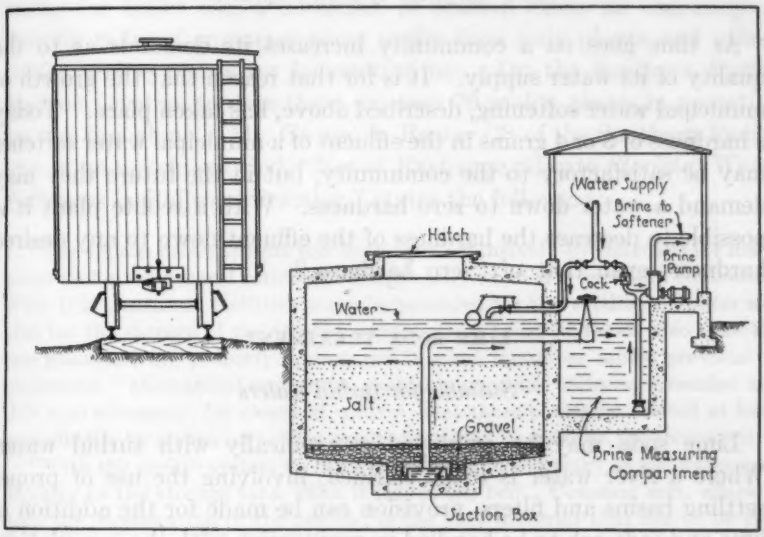

FIG. 2. WET SALT STORAGE SYSTEM

bottom keeps the undissolved salt from passing through and also provides some filtering action for the brine.

Unskilled labor may operate zeolite plants

This is a matter of special importance in the average small community where the labor cost appreciably affects the total cost of operation. With a manually operated zeolite plant, the labor required is merely in the regeneration of the units. Unskilled labor accomplishes this satisfactorily. With an automatic zeolite softener, as will be described below, even this labor of regeneration is saved.

With a lime soda plant, unless the water supply is of absolutely

constant composition, it is hazardous to employ unskilled labor in the operation of the plant. Even well supplies change in composition from season to season and from year to year and with unskilled labor the tendency is to use the same amount of chemicals and not to analyze the raw water and consequently, under or over treatment is possible. It is for that reason that, in making up comparative estimated costs of operation of zeolite vs. lime soda, some provision should be made for the salary of a chemist in connection with a lime soda plant.

Low hardness of zeolite effluent

As time goes on a community increases its demands as to the quality of its water supply. It is for that reason that the growth of municipal water softening, described above, has taken place. Today a hardness of 3 or 4 grains in the effluent of a municipal water softener may be satisfactory to the community, but in the future they may demand a water down to zero hardness. With a zeolite plant it is possible to decrease the hardness of the effluent down to any desired hardness, even that of "zero hardness."

LIME SODA TREATMENT

Suitable for turbid waters

Lime soda may be employed economically with turbid water. Where a river water is to be clarified, involving the use of proper settling basins and filters, provision can be made for the addition of lime and soda ash to be handled in conjunction with the coagulation. It must be pointed out, however, that for coagulation of turbidity a relatively small settling basin may be sufficient, of 2 to 3 hours capacity, whereas with lime, settling basins having a detention period of 4 to 6 hours should be supplied. Furthermore, with a coagulation plant carbonation equipment is not required, whereas with lime soda provision for carbonation should be included.

As a rule the small and medium-size community, however, draws its water supply from wells, in which case no preclarification is required, so that there would be no existing settling basins, filters, etc. available, in which the lime and soda ash might be economically employed.

Lower total solids in effluent

Lime treatment reduces the bicarbonate content of the water without forming any corresponding sodium salts in the effluent. Many prefer lime soda-treated water for boilers on that account.

In this regard it is to be noted that the sulphate or chloride hardness of the water, as treated by soda ash, does introduce a corresponding sodium salt content. The reaction between the soda ash and the sulphates forms sodium sulphate in the same way as with the zeolite process.

As regards the preference of some engineers for a lime soda-treated water for boiler use, a difference of opinion exists on this subject. Among railroad engineers some prefer lime soda plants and others prefer zeolite plants for locomotive use. On the Southern Pacific Railroad system to date there are over 70 zeolite plants in operation on this line alone. Mr. George L. Baxter (3) of the Southern Pacific line in an article entitled, "Use of Exchange Silicate (Zeolite) Water Softeners in Railroad Practice," states the following:

"The lime-soda treatment has been used extensively by railroads for many years and gives general satisfaction, especially when it is carefully controlled. With this treatment facilities must be provided for the settling of water and also for the storage of the treated water. Care must be taken also that the raw materials are properly housed and in some instances mixed previous to treatment. Mechanical apparatus, such as pumps and stirrers, is needed and it is also necessary, for chemical control, that these plants be visited at least periodically by a chemist so that exact check may be made on the treatment.

"With the zeolite system no tank for settling is necessary as the water goes directly to the storage tank when it leaves the bed. Common salt, which is used for regeneration of the zeolite, is the only material that has to be furnished. This material is unloaded directly from bulk shipment in cars to a large concrete tank, where it is dissolved in water and from which it is pumped to the zeolite bed when regeneration is necessary. Regeneration and washing, accomplished by the manipulation of valves, is all the work required for the maintenance of the zeolite plant. This regeneration means that the calcium and magnesium which have been eliminated from the water are in turn replaced by the sodium of the salt. The regenerating solutions and subsequent washings can generally be readily disposed of. This is quite a feature, as at some points on our lines we have no place to dump the sludge that is formed with the lime-soda treatment and the handling of this sludge also is an operation in itself under the best of conditions.

"It is therefore apparent that the zeolite method is practically 'fool-proof' as compared with the lime-soda treatment. Furthermore, it does not require attention due to the seasonal changes in water, as the zeolite itself takes care

of any variation in the chemical constituents of the water, whereas the lime-soda method may at times require considerable changing."

The zeolite installations made by the Southern Pacific Railroad are not confined to waters of low hardness. Table 4 gives the analyses of the raw water and capacities of typical zeolite plants on the Southern Pacific Railroad. It is to be noted that the alkalinity is

TABLE 4

Analyses of raw water and capacities of typical zeolite plants of the Southern Pacific Railroad Company

NUMBER	STATION	PLANT CAPACITY, GALLONS BETWEEN REGENERATIONS	SOURCE OF SUPPLY	GRAINS PER GALLON	
				Hardness	Alkalinity
3	Oakland, Calif.	120,500	City	16	7
5	Tracy, Calif.	75,000	Well	20	8
8	Surf, Calif.	53,500	Spring	22	13
9	Niland, Calif.	42,000	Ditch	28	10
11	San Jose, Calif.	83,500	Well	14.6	12.5
12	Davis, Calif.	15,000	Well	38	29
16	El Paso, Texas	125,000	Well	12	10.5
17	El Paso, Texas	116,666	Well	13.3	10.6
19	Victoria, Texas	47,500	Well	25	17.5
21	Skidmore, Texas	50,000	Well	18	12.5
24	Pettus, Texas	28,000	Well	25	14
33	Waelder, Texas	67,200	Well	20.2	11.5
35	Houston (Shops), Texas	168,000	Well	9.3	11
37	Houston (Roundhouse), Texas	126,000	Well	8.25	10.7
39	San Antonio, Texas	134,000	City	13.1	11.6
51	Lafayette, La.	180,000	Well	5.3	6
57	Ennis, Texas	106,000	City	4.5	5
64	Carlin, Nevada	105,000	Wells	12.5	13
65	Wells, Nevada	87,500	Wells	13	13.8
66	Los Angeles, Calif.	128,000	Wells	19.3	13.5
71	Fresno, Calif.	166,000	Wells	3.0	4.0

high in a number of these cases and nevertheless zeolite treatment has been preferred. It is also to be noted that in a few of the plants hardnesses as low as 4 to 5 grains in the raw water have been considered advisable to soften, so that the locomotives would get the benefit of "zero hardness" water.

In the industrial field also, zeolite treatment has made great progress for stationary boilers. Many of the large oil refineries, paper

mills, power plants, steel mills, tobacco companies, chemical industries have installed straight zeolite softeners to soften waters containing considerable alkalinity. About 50 percent of the total sales of industrial zeolite softeners expressed in dollars is for the softening of boiler feed water.

Corrosion prevention

It is often stated that lime soda-treated water protects the city distribution system from corrosion. Due to the fact that lime soda-treated water usually contains some free hydrate and carbonate which cause after-reactions, these deposits act as protective films in pipe lines and prevent oxygen from corroding the steel piping. However, with the advent of carbonation this condition changes to a considerable extent.

In the Ninth Annual Report of the Ohio Conference, October, 1929, page 36, evidence of corrosion in piping using municipally lime soda-treated water is given as follows:

"Mr. Campion, supervising chemist for Leroy, had some sections of galvanized iron pipe which had been used in the hot water system of one of the consumers. This pipe showed deep pits and some perforations. The Leroy water is softened by the lime soda process and always contains considerable normal carbonate alkalinity. It would naturally be expected to have incrusting rather than corrosive qualities. Up to the present time he can give no remedy for the condition.

"Mr. Van Arnum of Youngstown, is experiencing the same condition in hot water piping caused by water treated with excess lime.

"Mr. Rupp of Oklahoma City, Oklahoma, said that they are having trouble with corrosion there also. This is evident in action on the brass in some of the meters. Where galvanized nipples are connected to meters corrosive action, probably due to electrolysis, is taking place, necessitating replacement of the nipples in many cases in less than one year. He said he had no remedy to offer."

The causes and prevention of corrosion in municipal distribution systems have not yet been completely worked out. In some plants where lime is being added after a filter plant for the production of protective films, it has been found that these films were formed at the inlet end of the distribution system but were not protecting the more distant ends.

It is possible to produce protective films in municipal distribution systems using zeolite softened water by introducing soda ash or sodium silicate or both by means of a simple pressure solution feed.

At Wapakoneta, Ohio, where there is a zeolite plant, no corrosion has been experienced as evidenced by table 5 which gives the analyses of the water taken from a number of points throughout the city.

However, when a zeolite plant is first installed on a raw water that previously contained considerable amounts of iron, it is sometimes found that the old iron deposits in the city distribution system may contribute a slight amount of iron to the soft water for a certain period of time. This is especially true if the water is free from oxygen, because oxygen-free waters have been known to reduce old iron deposits and have cleared up piping systems that were

TABLE 5

Analyses of zeolite municipally treated water from various points in Wapakoneta, Ohio, November, 1931*

Results in p.p.m.

	UNTREATED	FROM ZEOLITE SOFTENER	WATER ENTERING DISTRIBUTION SYSTEM	CITY BUILDING	STOCK YARDS; EXTREME SOUTH	14 GLENWOOD AVENUE
Total hardness as $CaCO_3$	393	4	63	63	61	60
Alkalinity as $CaCO_3$	338	319	343	343	346	340
Turbidity	20	1	1	1	5	2
Color	20	5	5	5	5	15
Total iron	1.8	0.1	0.2	0.2	0.8	0.15
pH value	7.2	7.1	7.3	7.3	7.4	7.3

* Made by Department of Health, Division of Laboratories, Columbus, Ohio.

previously clogged with iron oxide deposits. This has been the experience in a number of deaerator installations in industrial practice.

AUTOMATIC ZEOLITE WATER SOFTENERS

Due to the fact that the only labor involved in the handling of a zeolite plant is that required in the regeneration of the softeners, a number of attempts have been made to design an automatic zeolite softener that would regenerate itself, thus saving the cost of labor involved in the handling of the apparatus.

Mr. E. T. Turner of Dayton, Ohio installed the first municipal automatic water softener for which he obtained U. S. Letters Pa-

tents.[2] He made the installation at Cedarville, Ohio which has been in successful operation since January, 1931.

An outline of this Cedarville installation is given in figure 3. The plant consists essentially of two duplex gravity concrete softeners with mechanical means for automatically regenerating it at proper intervals. It includes a soft water storage basin of suitable size and a wet salt storage basin sufficiently large to hold a carload of salt. The entire plant, including the soft water basin, wet salt storage basin, softeners, pump pit, and automatic means for starting and

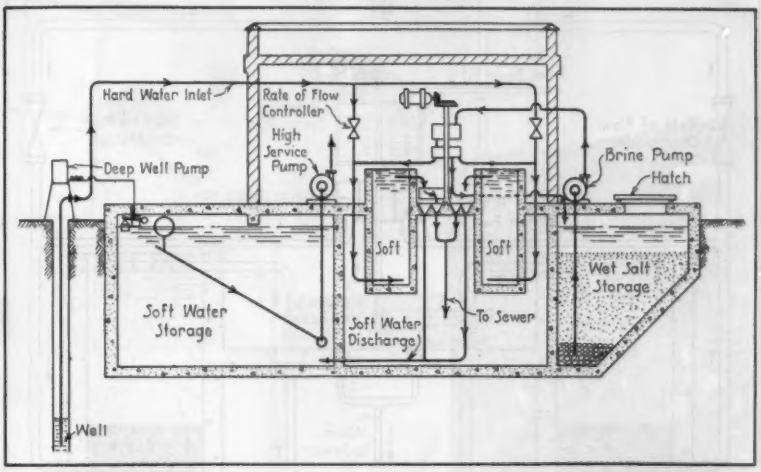

FIG. 3. AUTOMATIC WATER SOFTENER AT CEDARVILLE, OHIO

stopping the operation of the pumps and softeners at proper intervals, is included in one building.

The general design of the mechanism which automatically operates the softeners is shown in figure 4. A motor, through a set of speed reducing gears, rotates the two pans: the upper pan is the brine pan and the lower is the rinse pan. The brine is pumped continuously, by means of a small brine pump, into the brine pan and flows back to the wet salt storage basin if no regeneration is being carried out.

When one of the units needs regeneration, the rotation of the brine pan brings the pan's outlet over that part of the brine receiver below

[2] The General Water Treatment Corporation, of which The Permutit Company is a subsidiary, has an exclusive license under these patents.

it which leads to the hard water inlet of the softener. In that way brine is diverted to the softener that must be regenerated. The speed of rotation of the brine pan and the design of the brine receiver is arranged so that, after a definite interval, the brine is again diverted back to the salt storage basin. This controls the amount of brine introduced into the softener. During the regeneration, the incoming hard water continues to flow at a constant rate, regulated by a rate of flow controller. The brine is of saturated strength and, when it

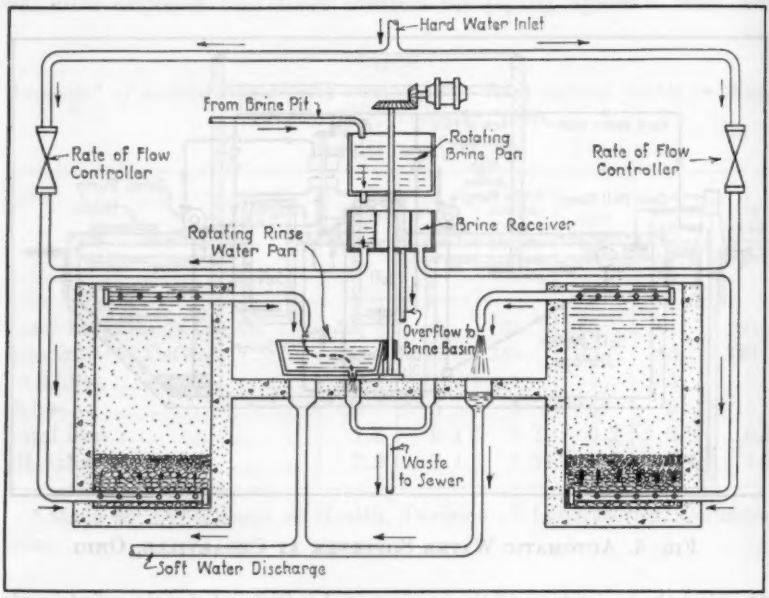

FIG. 4. AUTOMATIC GRAVITY ZEOLITE SOFTENERS

mixes with the hard water, is diluted to the strength that gives the best regeneration.

The diluted brine then flows through the zeolite and escapes into the rinse pan and thence to the sewer, carrying out with it the calcium and magnesium chlorides removed from the zeolite during the regeneration process. This rinsing operation continues via the rinse pan until the brine has been rinsed out of the zeolite. At that point, the rinse pan is automatically rotated away from the outlet of the softener, so that the softener discharges direct to the pipe leading to the soft water storage basin.

A view of the mechanism itself, showing the brine receiver and a vertical shaft leading down to the rinse pan, is shown in figure 5.

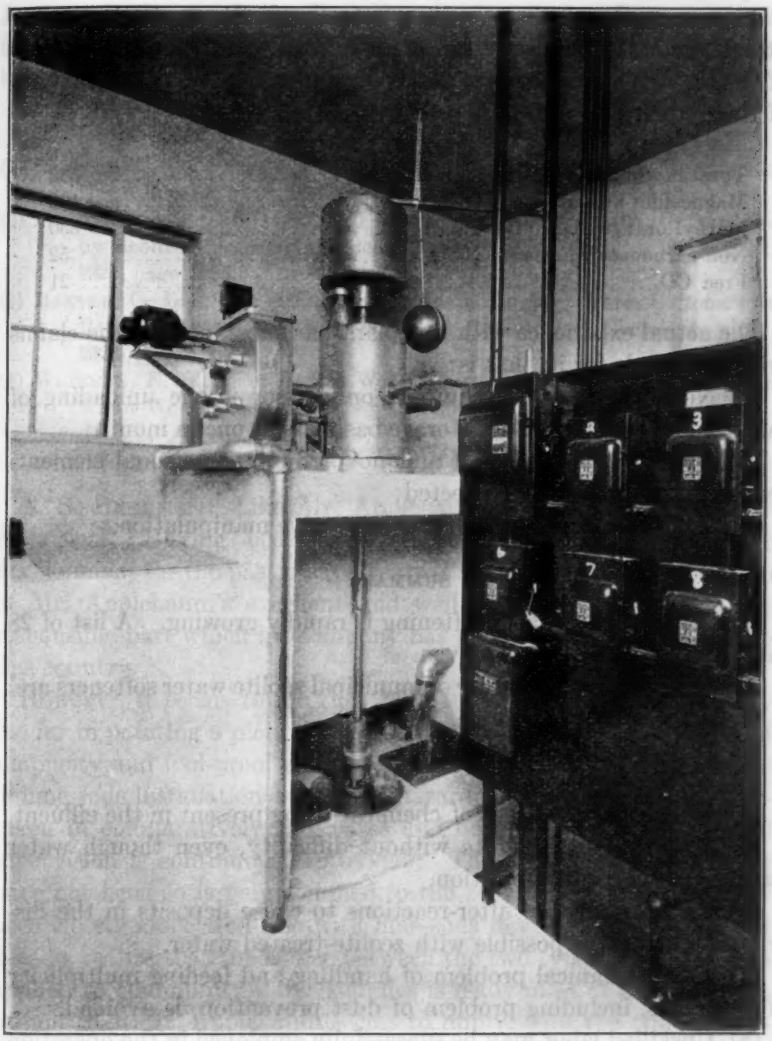

FIG. 5

It is apparent, therefore, that the automatic mechanism consists essentially of a motor-driven shaft with two pans mounted thereon.

This arrangement takes the place of any operating valves on the softeners. The entire mechanism is therefore extremely rugged and simple.

At present the city is operating only one of the duplex units, the other being held in reserve for future needs. The ultimate capacity of the plant is approximately 200,000 gallons per day. The analysis of the Cedarville water is as follows:

	p.p.m.
Total hardness as $CaCO_3$	382
Magnesium hardness as $CaCO_3$	100
Methyl orange alkalinity $CaCO_3$	330
Non-carbonate hardness $CaCO_3$	52
Free CO_2	21

The actual experience with this plant has substantiated the claims originally made for it, that is:

(a) No supervision is required—only labor is the unloading of salt from truck into wet salt storage basin about once a month;

(b) Operation is fool-proof and not subject to personal element-regenerations cannot be neglected;

(c) Equipment operates without any valve manipulation.

SUMMARY

Municipal zeolite water softening is rapidly growing. A list of 28 municipal zeolite plants is given.

The considerations in favor of municipal zeolite water softeners are:

(1) Absence of any sludge.

(2) Low initial cost.

(3) Low operating cost.

(4) No danger of excess of chemical being present in the effluent.

(5) Zeolite plants operate without difficulty, even though water supply changes in composition.

(6) No possibility of after-reactions to cause deposits in the distribution system is possible with zeolite-treated water.

(7) The mechanical problem of handling and feeding multiplicity of chemicals, including problem of dust prevention, is avoided.

(8) Unskilled labor may be successfully employed in the operation of zeolite plants.

(9) Hardness of effluent may be as low as desired.

The considerations for municipal lime soda treatment are:

(1) Suitability for turbid waters.

(2) Lower total solids in effluent.
(3) Corrosion prevention.

A new automatic municipal zeolite water softener installed in January, 1931 at Cedarville, Ohio is described. This saves all labor required in the regeneration of the water softener, the operation being conducted mechanically and automatically.

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DISCUSSION

A. S. BEHRMAN:[3] Like Mr. Applebaum, I am an ardent zeolite enthusiast. Having been rather closely associated with zeolite development for the past twelve years, I have a genuine appreciation of Mr. Applebaum's excellent and well-written paper, and of the responsible part which his company has played in the zeolite field in this country.

However, it seems to me that Mr. Applebaum has gone entirely too far in painting a picture of a zeolite water softener as an ideal of simplicity and fool-proof efficient operation, and, in contrast, a lime or lime-soda installation as rather a hazardous enterprise to be undertaken in comparatively rare cases and only when skilled chemical supervision is continuously provided. Those of us whose interests have not been so largely confined to the zeolite field, and who have been closely associated also with lime-soda developments in the last decade, realize that the choice between the two types of softening as applied to community water supplies is really more of a 50–50 proposition than Mr. Applebaum seems to imply, with the odds if anything in the majority of cases in favor of lime treatment.

It is true that salt is pleasanter to handle than lime. It is equally true that, generally speaking, the operation of a zeolite softener is

[3] Chemical Director, International Filter Company, Chicago, Ill.

simpler than a lime-soda softening plant. There is no gainsaying the fact that in some cases the disposal of sludge is an important problem, and, in a few cases, a determining consideration.

On the other hand, it must be remembered that there are some types of waters which because of their chemical composition—not because of any turbidity which Mr. Applebaum mentions as one of the few factors favoring lime treatment—can be softened by the zeolite process only at prohibitive initial and/or operating expense.

Among such waters may be mentioned those that are extremely hard, and those which contain large amounts of sodium salts. The volume of zeolite required in zeolite softening increases almost directly with increasing hardness of a water. In lime-soda softening the size of the installation is determined by the hourly demand for soft water; the size is not affected by the hardness of the water, nor is the capacity diminished by unlooked-for increases in hardness.

Similarly, when waters are high in carbonate hardness, it is usually much cheaper to soften them with lime than it is to supply the salt necessary for softening them by the zeolite process. In some cases, however, particularly in small communities, this saving in chemical cost is more than balanced by some of the operating advantages of the zeolite process which Mr. Applebaum has discussed; but this situation does not usually obtain with installations of reasonably large size.

Another important and very generally applicable consideration is the question of corrosion with zeolite-softened water; and we have too much information on this point to dismiss the matter as lightly as Mr. Applebaum has done. At New Bremen, Ohio, for example, at which is located the zeolite plant described by Mr. Applebaum as successfully removing iron and hardness simultaneously, there have been steady and serious complaints of red water, arising from corrosion of the distribution system. Prevention of corrosion of distribution lines with zeolite softened water can be corrected in some cases by mixing with the softened water a certain proportion of hard water, and, in others, by adding chemicals to put the water in condition to deposit a protective coating on the metal. The fact remains that a zeolite-softened water is a potentially corrosive water, the corrosive tendencies of which must be recognized and, if possible, corrected. A lime-softened water, on the other hand, is from its very nature an essentially protective one, Mr. Applebaum's exceptions to the contrary notwithstanding.

The question of wash water consumption with zeolite softening plants is a consideration which is frequently overlooked. The amount of wash water required during the regeneration cycle is a function primarily of the area of the softener. If the hardness of the raw water is high, and if a zeolite of low base exchange capacity—such as greensand—is employed, the wash water may easily amount to as much as 25 percent of the volume of water softened. We have been able to remedy this situation materially in the past ten years by developing manufactured zeolites having three to four times the base exchange capacity of greensand, and capable of exchanging their bases at very rapid rates of flow. In this way we are able to reduce the area of the softener and to lessen the frequency of regeneration, resulting in a very considerable saving of wash water—averaging about 50 percent—of the wash water required for a greensand softener of equivalent capacity. While this improvement is obviously a most desirable step forward in economical operation, it must not be forgotten that in lime-soda softening plants the wash water consumption is commonly well under 5 percent.

Reasonable intelligence is required for reasonably efficient operation of a zeolite plant. The same intelligence will as efficiently operate a lime treatment plant also. I am sure that a skilled chemist is not provided for the operation of every small municipal lime-softening plant in the country, or for even a reasonable portion of them; and in this connection I cannot help but think of the upwards of 4000 lime treatment plants our company has installed in raw water ice plants alone, where the softening treatment is controlled by the operating engineer. Not that I would deprecate for a moment the employment of capable chemical supervision for either a lime-soda or zeolite plant. On the contrary, I know from experience that skilled chemical supervision will result in such improved operation that the investment in a competent chemist for either occasional or full-time supervision will pay excellent dividends. My point is that the advantage of such supervision is not confined to the lime-soda plant.

Of particular interest to me is Mr. Applebaum's description of the automatically operated zeolite plant. Household zeolite softeners operated automatically have been used to a certain extent for a number of years. Where all conditions are substantially constant, softeners of the automatic type possess certain desirable labor-saving characteristics, though having also certain definite disadvantages.

When the hardness of the raw water fluctuates, or when some other unlooked-for disturbing condition arises, it is obvious that an automatic softener cannot, from its very nature, operate efficiently under the changed conditions.

In short, while I am frank to confess a great personal enthusiasm for zeolites and zeolite softening, and believe that its employment for treating municipal supplies will expand—as will expand the softening of municipal water supplies in general—I feel that we should temper our enthusiam with the proper appreciation of the limitations, as well as the advantages, of the method.

To those who are interested in going further into the determining factors involved in the choice between lime-soda and zeolite softening for municipal supplies, I would recommend the very thorough and sane analysis of the subject presented by Hughes and Crane before the Association at the Toronto meeting, and published in the January, 1930, issue of THE JOURNAL.

J. F. LABOON:[4] The paper which Mr. Applebaum has presented gives a thorough analysis of the relative merits of zeolite and the lime-soda softening processes and especially does it make out an excellent case for the zeolite type of softening plant.

Softening by the zeolite process on a large scale is no longer an experiment, as we have demonstrated in two plants on the Ohio River where water from wells and cribs is softened; namely, Ohio Valley Water Company at McKees Rocks, Pa., and the Sewickley, Pa., Municipal Plant. Another municipal plant which we have designed and which is now in operation, of the zeolite type, is at Oakdale, Pa., where well water is softened. However, the selection of zeolite type for these three plants was determined by the economies involved in the problem and the choice was made in favor of zeolite only after a thorough study of capital expenditures and operating expenses of both the zeolite and lime-soda processes. Likewise, local conditions, after a full study of the economics of the case, should determine what process is best suited, from the standpoint of results to be striven for.

One of the fundamental differences between the zeolite and lime-soda processes is this—in the zeolite process the total solids content, that is, the total amount of chemicals in the softened water is practically unchanged, whereas in the lime-soda process a portion of the

[4] Of The J. N. Chester Engineers, Pittsburgh, Pa.

chemical solids content depending upon the carbonate content is removed. The removed chemicals take the form of sludge, so-called and this is one of the most serious problems of the lime-soda process. Sludge in this process is voluminous and generally creates a nuisance, mainly from the standpoint of aesthetics when discharged into an open stream. As yet, while many theories have been advanced for the disposal of this sludge, the problem remains mainly unsolved. Local conditions again may permit the storage and disposal of slugde in great quantities for many years without creation of nuisance.

The zeolite process has very little competition where the raw water from streams, wells, or cribs is constantly clear and where the hardness is not too great to produce an objectionable concentration of chemical solids where the softened water is used in boilers for steam purposes. Excessive foaming and blowing off of the sludge formed in the boilers, due to too high concentration of solids, may prove to be a vital factor in the selection of the respective type of softening treatment.

From our experience the open type gravity zeolite softener is much preferable to the pressure type, inasmuch as the latter permits very little insight into the condition of the filters and the successful operation of the process. We have found that raw water taken from wells and river cribs may develop manganese problems which aggravate the zeolite process and especially the pressure type of tank. The whole story has not as yet been told about manganese.

The Sewickley municipal installation, which is of an open filter gravity type and designed for both filtration and softening of raw waters drawn from cribs located in the Ohio River, has justified the installation of the zeolite type, as measured by results and operating costs, including fixed charges on the capital investment. Manganese troubles developed at Sewickley, but this problem is being solved.

Corrosion in hot water systems is caused, generally, by the liberation of dissolved oxygen and other gases which are driven off under higher temperatures and, therefore, it is common to both the lime-soda and zeolite processes and is no particular property of either. That this has been recognized by engineers is evident from the fact that deoxygenation plants are in general use in the larger installations of hot water systems.

The lime-soda softening plant which we designed for Fostoria, Ohio, has operated intermittently, primarily because softening costs money and many of the municipalities cannot afford it today, as

measured by their budgets. However, sludge disposal also is a problem at Fostoria, although they have a capacity for several years storage and an arrangement has been worked out whereby the stored sludge may be pumped into the river when it is high and thus prevent nuisance and at the same time provide convenient disposal of the sludge.

S. B. APPLEBAUM: Mr. Behrman's discussion of my paper on Water Softening by Zeolite as Used in Municipalities adds information of considerable value in connection with a thorough appreciation of this subject of municipal water softening. We agree with Mr. Behrman that there are many waters that are suitable for lime soda treatment and it was not the intention of our paper to imply that zeolite is the only form of water treatment that should be employed for municipal plants.

Among the disadvantages of zeolite which Mr. Behrman enumerates, he implies that when the hardness of the water changes the zeolite softener may become too small for the installation. This is not in accordance with the facts, because by changing the number of regenerations to be given a zeolite softener daily, considerable flexibility is provided. If a zeolite softener is regenerated two times a day with 20 grains hardness and the hardness of the water should be increased to 30 grains, the same zeolite softener without any change in the volume of zeolite used, can handle the water of higher hardness by merely regenerating three times instead of twice a day.

Secondly, Mr. Behrman pointed out that in a great many cases the operating cost and first cost of zeolite softeners exceed that of lime soda softeners without giving illustrations to prove his point. In the paper I have given several cases showing the reverse, with waters that should be considered typical municipal well supplies, showing that the first cost and operating cost with zeolite softeners are usually lower.

Mr. Behrman states that zeolite softened water is a potentially corrosive water, whereas lime softened water is essentially a protective water. No facts are given again to substantiate these bare statements, whereas in the paper we have given actual evidence showing the reverse. The reference made to New Bremen disregards the fact that in the city mains there was a very large accumulation of iron oxides previously deposited from the use of this heavily iron-laden well supply for many years standing. These iron oxides are slowly being removed by water low in oxygen, which causes the

condition in question. A sodium silicate feed is now being installed at New Bremen to cover up these old iron oxide deposits to prevent their being further leached out by a water low in oxygen. We believe the statement that zeolite softened water is a potentially corrosive water is entirely unsupported by the actual evidence. In the paper we have given results at the City of Wapakoneta and no complaint of corrosion has been made by any of the other many municipal zeolite plants installed. Furthermore, there are undoubtedly over 10,000 industrial zeolite plants in this country and the experience of these many plants confirms the successful experience of municipal plants in this regard.

A further disadvantage of zeolite mentioned by Mr. Behrman is the high percentage of wash water. If wash water is a matter of any importance, it is usually possible to recover most of the wash water and use it over again. This is being done by a great many industrial plants where water is scarce. It is true that synthetic zeolites may require less wash water than Glauconite and where the water is suitable for synthetic zeolites they should by all means be used. With upflow synthetic softeners, however, the presence of iron in the well waters calls for iron removal filters in advance of the synthetic softeners, to insure proper iron removal and to prevent clogging of the synthetic zeolite. In the case of Glauconite downflow softeners, such iron removal in advance of the zeolite is avoided.

EXPERIENCE WITH ACTIVATED CARBON

By George R. Spalding[1]

When we review in broad general terms the history of water purification as an accomplishment of relatively recent times we find that there have been two major contributions to the science each of which has been of far reaching significance. First of these was filtration, which came as necessity brought about the use of surface supplies carrying at times unsightly color or turbidity.

The average consumer expects a pure water supply to be clear, colorless and sparkling and judges the quality of his drinking water largely by these characteristics. Filtration, in its many and diversified forms, has solved this fundamental problem and given him a thoroughly acceptable product when considered from this standpoint alone.

Then came the second milestone in water treatment: chlorine sterilization; an adjunct of unquestionably greater importance from its influence upon public health than any other single agency. Many of us will recall the feeling of security that was brought to purification plant operators when the use of chlorine was introduced nearly twenty-five years ago.

Have we now reached the third important fundamental step in the production of thoroughly acceptable water for general public consumption wherein the sources of supply are largely of surface origin. Is the answer yes and is the medium by which that is to be accomplished "Activated Carbon?" Have we arrived at the point where we shall be able to produce water that is not only satisfactory in appearance and bacteriologically safe, but also palatable?

If you are a carbon enthusiast, perhaps the answer is yes. If, on the other hand, you have watched with a feeling akin to dismay, the increasing group of chemicals that have been adopted as water purifying agents you may be inclined to view this newest one as just another much heralded panacea for all water evils. Certain it is that no method of water treatment has aroused more interest and

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discussion in recent years than has been accorded to activated carbon.

In reviewing experiences with carbon in the pulverized form, it is apparent that this substance would find its greatest field of usefulness in that class of waters which requires filtration and presents continuous or occasional problems in odor and taste control. This field embraces particularly surface waters carrying contaminating substances in some degree which form the sources of supply for many of the large centers of population.

Such surface supplies receive an extraordinary variety of contaminating materials when we stop to reflect upon it.

Nature herself furnishes a long list of substances principally of vegetable origin which are continually undergoing their life cycles. As a result of this growth there is quantity production going on of such things as sugars, vegetable oils, organic acids and esters, resins and waxes, many of which finally return to the soil. Disintegration processes are continually under way which impart odors or tastes to water secured from catchment areas, while the flora of ponds, lakes and swamps contribute an endless variety of seasonal odors and tastes which we classify by such terms as earthy, woody, vegetable, bitter, sweet, aromatic and so forth. Every rainfall that washes the uplands carries these substances into the water, producing therein characteristic surface water odors.

In addition to the natural and characteristic odors that waters develop there is an every increasing number of contaminating substances of man made origin. Particularly in those surface supplies derived from populated areas the list of such artificial polluting material is a long one of great diversity and is increasing.

The automobile has penetrated into most remote places carrying with it a trail of oily wastes and the general use of tarry compounds in highway construction has introduced a particularly potent contaminating substance.

Wastes from a great variety of industrial processes have found their way into water supplies with increasing frequency and the number of highly odorous chemicals that are potential sources of trouble in potable waters has increased greatly in the last quarter century.

It is probable that prohibition has caused considerable increase in the amount of wastes from alcoholic fermentation and distillation processes that find their way into streams which are used as sources of water supply. Such fermenting wastes are capable of conveying to water a particularly malodorous contamination.

Wherever pollution of streams occurs from *human wastes* either directly or through the medium of the various types of sewage purification processes, still another group of substances is introduced which contribute to the sum total of tastes and odors. This group includes the nitrogenous organic compounds in various stages of bio-chemical decomposition. Detailed discussion of the variety and character of such sewage-borne contaminating substances is here unnecessary. It is pertinent to point out that some of them have presented insuperable obstacles to the various removal processes that have been available to waterworks chemists.

If I were to go on enumerating the various things that cause odors and tastes in water supplies, I could but accentuate the fact that the tremendous diversity of these contaminating substances renders the problem of their removal a difficult one. The waterworks chemist deals with well established practices in the field of purification except in his efforts to produce "palatable" water and at that point the number of intangible factors offers serious obstacles to standardized methods.

Beyond a doubt therein lies the dominating appeal, the promise of a hope fulfilled, that has set many waterworks men eagerly to the task of trying out this new adsorptive agent, "activated carbon."

The fact that it brings about its action by purely physical means, adding nothing of a chemical nature to the water and depending not at all upon mere chemical alteration of odor producing substances serves to give it added value.

And when I speak of this *new* adsorptive agent "activated carbon" this term should be qualified as the only really new thing about it was the discovery of a simple and suprisingly effective method of applying it to water in the form of a fine powder.

Abundant time has elapsed since the announcement of this process was made at the St. Louis Convention in 1930 to secure data covering the use of carbon by a great many workers under widely diversified conditions. Their work has established beyond doubt the efficacy of surprisingly small amounts of this agent in removing those tastes and odors that commonly spell the difference between really inviting water and a product that the average consumer can find fault with.

The experience of these plant operators cover a wide range of conditions with waters in various parts of the country. Odors and tastes due to algae and decaying vegetation are removed particularly well and at economical cost. Such substances as phenols, hydrogen

sulphide, and petroleum products have been successfully removed and one worker reports that carbon removes the alkali taste produced in water softening.

NEWER DEVELOPMENTS

At the Plant of the Hackensack Water Company we applied an average dosage of 9.7 pounds per million gallons during a total of 249 days in 1931 to control seasonal odors (principally) from microscopic organisms such as Synedra, Melosira, Asterionella, and Synura.

Many operators report that the use of powdered carbon in conjunction with the alum coagulation process reduces the amount of of alum necessary and gives better flocculation particularly at low temperatures. Better coagulation minimizes the tendency for free albumina to pass through into the distribution system where it causes trouble.

Others point to a definite reduction in the chlorine demand of water pre-treated with carbon and it appears well established that carbon does remove effectually those unstable organic substances in solution which are the first subjects of oxidation by chlorine. Removal of such substances doubtless accounts in part for the increased efficiency of alum coagulation and consequent decrease in alum dosage necessary. The fact that activated carbons are effective decolorizing agents explains why small amounts assist materially in color removal by the coagulation process.

Carbon treatment has been extended to the deordization of new water mains to remove tarry odor following the usual sterilization with chlorine before the mains are placed in service. All that is necessary is to introduce the carbon in the form of an aqueous suspension, fill the main with water, allow a contact period of several hours and finally flush the main vigorously to remove the carbon.

An outstanding feature of the action of carbon which has been observed by numerous investigators is its ability to inhibit the process of putrefactive decomposition of organic solids which takes place in coagulating basins. Experience at the purification works of the Hackensack Water Company shows conclusively that continuous application of less than ten pounds per million gallons of pulverized carbon with the alum coagulant produces an entirely stable sludge with no tendency to become odorous. Nor does the stabilizing effect terminate at this point, as we have found that freedom from

odor extends to the filters and even to the filtered water storage wells. These wells are used as lime treatment reservoirs for removal of free carbonic acid and correction of the pH value of the water delivered to the mains. Previous to the use of activated carbon in the coagulation process the lime sludge in these clear water reservoirs developed an odor noticeable when the wells were cleaned. After the carbon process was introduced this lime sludge was observed to be entirely non-odorous.

I consider this inhibitive effect of carbon of very favorable significance as the best practice in rapid sand filtration requires a condition of maximum hygienic cleanliness throughout the successive stages of treatment and activated carbon furnishes a valuable means to attain this end.

No explanation of this inhibitive action has come to my attention and the subject presents a problem for investigation. It is quite possible that carbon removes from the liquid medium proteins or other substances essential to the development of bacterial life.

CONTROLLING CHARACTERISTICS OF CARBON

Eliminating in this discussion the highly theoretical conception of the principle of adsorption which depends upon differences in electrical potential of the adsorbent (carbon), the material adsorbed and the liquid medium, there are a number of characteristics of activated carbon which affect its efficiency from a practical standpoint. Inasmuch as adsorption is a function of the surface of the carbon particles it is obvious that the factors influencing carbon efficiency are (1) actual carbon content of the material (2) the degree of activity (3) fineness of the particles and (4) uniformity of fineness of the particles.

Mr. James E. Kerslake of the firm of Nicholas S. Hill, Jr., Consulting Engineers, has furnished table 1 showing the relative surface area of carbon particles which will pass standard mesh sieves ranging from 200 to 325 meshes per inch. Table 1 is based upon the assumption that all particles are perfect spheres and no allowance was made for voids.

From Mr. Kerslake's figures it is apparent that the unit mass of a very fine carbon would present materially greater surface for adsorptive effect and ought to exhibit greater efficiency other factors being equal. Actually a full scale test of carbon passing a 300 mesh sieve made at the Hackensack Water Company Plant in 1931 failed

to show any material increase in efficiency over the commercial grade which is ground to pass a 200 mesh sieve. While this observation may not be conclusive there is reason to believe that the factor of extreme fineness is of relatively minor concern.

Most important, in my opinion, is the method of application of pulverized carbon in such a manner as to secure extremely intimate distribution of the carbon particles throughout the entire mass of water being treated. To secure this thorough mixing and an adequate period of contact is essential whether the carbon be applied in the coagulation process, upon the filter beds or elsewhere.

TABLE 1

Relative surface areas of carbon particles

MESH PER INCH	OPENING INCHES			RELATIVE SURFACE AREA
200	0.0029	Passing	200	1.0
		Retained on	230	
230	0.0025	Passing	230	1.25
		Retained on	270	
270	0.0021	Passing	270	1.59
		Retained on	325	
325	0.0017	Passing	325	1.75
		Retained on	350	

Devices for the specific application of such voluminous and finely pulverized active carbons have not been available until recently and many workers have improvized homemade appliances. This is not altogether satisfactory as the problem of distributing the ordinary dosage of ten to twenty pounds of this fine powder absolutely uniformly in a million gallons of water cannot be met with makeshift devices. A number of manufacturers are now bringing out dry-feeding machines specifically designed to handle pulverized carbon.

The exact method of utilizing carbon to best advantage is, after all, something to be worked out specifically at each plant and depends largely upon local conditions.

Dismissing further enumeration of details in the use of carbon, which have been widely publicized by various workers, I cannot terminate this discussion without returning to consideration of the reasons for such widespread interest in this agent. And, as I see the

facts, carbon has focussed attention upon a problem not new or original, but of general concern and of increasing importance. That problem is how to give the public water that they like: that tastes good.

Perhaps I lay myself open to rebutal when I say that the general public is becoming more critical of its drinking water. There certainly is no question that they are becoming better informed on this subject, as they are in many other matters affecting public health.

It is also safe to assert that many waterworks chemists seeking to deliver a product that will satisfy their consumers have been sorely tried at times by their helplessness in eradicating unusual tastes and odors.

The problem is complicated by the fact that some taste and odor troubles originate in the water distribution system thus nullifying to a certain extent all prior corrective measures which may have been entirely effective.

One of the most distressing problems of this type which occurs in some water systems is a taste and odor described variously as tarry, carbolic or medicinal which develops in dead ends at summer temperatures due to water of a corrosive character. With water of this type, prolonged stagnation in iron pipes may result in complete removal of dissolved oxygen followed by a definite content of ferrous iron in solution and production of a disagreeable tarry taste and odor. This is a type of difficulty that fortunately can be largely corrected by suitable anti-corrosion treatment already practised in many water purification systems.

It is in elimination of those odors and tastes that hitherto have presented great difficulty and in some cases have baffled our efforts at removal that activated carbon makes a particularly strong appeal.

For emergency purposes in combating unusual and severe contaminating substances carbon will prove a boon to water plant operators.

Whether this new agent will take its place as an accepted adjunct to routine treatment depends entirely upon the attitude of the waterworks fraternity toward the problem of producing more palatable water—water of superior quality.

PRESENT STATUS OF AERATION

By Frank E. Hale[1]

Let us examine the possible functions of aeration and list them in order.

1. The removal or reduction of undesirable gases.
   a. Carbonic acid gas.
   b. Hydrogen sulphide.
   c. Excess chlorine.
   d. Possibly other gases such as nitrogen or marsh gas.
2. The addition of oxygen.
3. The removal of odors.
   a. Of decomposition, musty, moldy, or unpleasant.
   b. Due to microscopic organisms, aromatic, grassy, fishy, oily.
   c. Due to action of chlorine on organic substances, medicinal, carbolic, iodoform.
   d. Due to trade wastes.

The third section may be associated with the first section if the odors are due to gases, or the odors may be due to volatile essential oils or other volatile organic compounds.

4. Aeration as a part of other water purification processes, to
   a. Aid coagulation.
   b. Promote admixture of chemicals and water.
   c. Assist in washing the sand in filters.
   d. Help remove iron and manganese.
   e. Prevent red water.
   f. Freshen stagnant surface water.
   g. Break up colonies of fragile microscopic organisms.
   h. Assist in softening water.
   i. Assist in lime and iron sulphate treatments.
   j. Point of application in filtration processes:
      Applied to raw water.
      Applied after adding chemicals.
      Applied after filtration.
   k. For esthetic effect.

[1] Director of Laboratories, Department Water Supply, Gas and Electricity, New York, N. Y.

5. Methods of aeration.

a. Simple exposure to air, as in aqueducts, basins, and reservoirs.
b. Flowing over weirs in thin sheets.
c. Cascades by steps, descending troughs, or riffled slopes.
d. Trickling devices, stone or coke beds, perforated trays.
e. Sprays or fountains.
f. Blowing air through by pressure, through perforated pipes, strainers or porous plates.
g. Drawing air through, Aeromix.
h. Spraying water into air under pressure, Elgin process.

1a. Carbon dioxide may exist naturally in a surface or ground water, or it may be formed by chemical action as when alum is added. In a surface water it is highest during the late summer just below the thermocline and at the bottom, with considerable quantities in between, due to decay or wet combustion of organic matter or dead microscopic organisms. In our experience values of free carbonic acid have reached 11 p.p.m. just below the thermocline, 22 p.p.m. at the bottom, and 6 to 8 p.p.m. in between, at Kensico reservoir, with corresponding reduction of dissolved oxygen to zero just below the thermocline and at the bottom and 3 to 6 p.p.m. in between (1). These studies were made in 1916. In ground waters free carbonic acid is frequently very high ranging from 30 to 125 p.p.m. With alum treatment 1 grain per gallon of basic aluminum sulphate sets free on the average 3.5 p.p.m. carbon dioxide. It is sometimes stated that 7 p.p.m. is set free, but this statement is in error since the floc formed is basic carbonate of aluminum and not aluminum hydrate. This has also been proven by actual determinations (2).

The efficiency of the removal of carbon dioxide is a function of time, temperature, degree of exposure to the air, and rapidity of chemical actions producing carbon dioxide simultaneously as in oxidation of iron and manganese. The latter exist in solution as ferrous and manganous bicarbonates. In oxidation to ferric and manganic hydrates the carbon dioxide is set free.

The results of removal of carbon dioxide have been fairly well established and were well presented with regard to ground waters by Donaldson in Table 9, page 195, of the Manual of Water Works Practice (3). Values were reduced from 12 to 125 p.p.m. carbon dioxide before various types of aeration to 4 to 32 p.p.m. after aeration. The average percentage removal was 70 and varied from 43 to

90. Fourteen plants were represented. Four results with surface waters published by Pirnie in 1929 average 65 percent removal where values before aeration varied from 5.5 to 28 percent carbon dioxide.

1b. Hydrogen sulphide may be present in deep well water due to reduction from sulphate. It is usually present in small quantity and seems readily removed by aeration. Simple splashing removes it in the softening plant at Daytona, Florida, the water coming from a deep well. At Beverly Hills, California, the hydrogen sulphide is captured and discharged through a tall stack to avoid complaints in a residential section (4). At Dwight, Illinois, State Reformatory for Women, the hydrogen sulphide is removed by spraying into air under pressure in connection with an iron removal pressure filter, Elgin process. (Information by letter April 8, 1932).

However, hydrogen sulphide is very simply and completely removed by chlorination by maintenance of 0.05 to 0.10 p.p.m. residual chlorine. In the presence of considerable free ammonia with which chloramine forms, the reaction with hydrogen sulphide is secondary and in one instance to our knowledge required 0.6 p.p.m. residual chlorine before all odor of hydrogen sulphide was gone. This amount of residual chlorine, however, did not cause taste of chlorine due to formation of chloramine.

1c. Excess residual chlorine may be reduced by aeration. At Richmond, Virginia, and Rahway, New Jersey, spray nozzle aeration is effective after substantial pre-chlorination in reducing residual chlorine to non-troublesome limits (4). In connection with effluent chlorination it would seem preferable to place chlorination in final position to avoid possible contamination after the chlorination.

1d. Other gases such as nitrogen, marsh gas, etc., may be present in ground waters. We have noted abundant ebullition of gases when well water was pumped into a tank, water entirely devoid of oxygen and containing only 30 p.p.m. carbon dioxide. Complaints from consumers supplied with well water directly from the pumps have been received in some instances due to gas. In such instances where oxygen is absent and iron is low it is probably preferable not to aerate, in order to lessen corrosion of iron piping. Brass pipe, however, would be corroded if carbonic acid were high.

2nd. The addition of oxygen is a necessary accompaniment of aeration. With regard to corrosion its addition is a detriment. In iron removal it is essential in oxidizing the iron from ferrous to ferric condition, simultaneously with the loss of carbon dioxide.

Boiled water tastes flat. Lack of dissolved air is believed to be the reason. The addition of oxygen thus probably by its presence helps improve the taste of water.

3rd. The removal of odor is one of the chief functions of aeration. To determine the extent of benefit seems very difficult. Possibly the method recently proposed (17) by Spaulding, Springfield, Illinois, may eventually give us a definite measuring rod. Certain it is that a partial removal of odor must take place for the odors to leeward of an aerator are very distinct when odors are present in the water. Such removal is not complete, however. Aeration is simply helpful.

3a. Musty and unpleasant odors of decomposition are usually readily removed by aeration, at least such odors as result from stagnation below the thermocline in surface water and from anaerobic conditions in well water. Certain musty odors are due to decay of microscopic organisms when present in too small amount to give their characteristic odors.

Highly colored waters often produce very unpleasant odors in hot water supplies and may be the cause of disagreeable odor in the cold, particularly with chlorination. Aeration will not remove color nor the organic matters associated with it hence can hardly remove this source of odor.

There are certain musty odors due to other causes which apparently are not readily removed by aeration nor by many other available methods.

3b. Odors due to microscopic organisms, aromatic, grassy, fishy, etc. undoubtedly are greatly reduced by efficient aeration. These are probably due to volatile essential oils. The characteristic odors may be readily detected on the outskirts of an aerator. However, Synura oil is too persistent to be removed sufficiently by aeration even when present in small amount, though it readily disappears in an open reservoir following treatment with copper sulphate, i.e., after a few days. There is also a bitter taste with this organism which is very persistent and not removed by aeration.

3c. There seems to be general testimony that odors due to action of chlorine on organic substances, coloring matter, cold tar derivatives, etc., described as medicinal, iodoform, carbolic, are not removed nor reduced to any great extent by aeration. These odors sometimes disappear on standing several days, indicating either volatility or oxidation, hence they are doubtless partially affected by aeration.

3d. Odors due to trade wastes are of great variety and according to Donaldson no great help is obtained by aeration. Some assistance has been noted with petroleum wastes as noted by Donaldson and also Hansen (3) (5).

4a. In some instances aeration has been found to assist coagulation with alum and to lower the amount of chemical required. Pirnie gives as evidence the very soft turbid water at Danville, Virginia (five years experience at the time, 1929), and West Palm Beach, Florida, a soft, highly colored swamp water. In other cases aeration may be detrimental, require more coagulant and the floc be broken up.

There may be two points of view: The Aeration may be a rapid and excellent mixing device, or it may regulate the hydrogen-ion concentration. The amount of carbonic acid is a potent factor in the latter respect. Colored soft water coagulates best usually at a low pH, other waters generally nearer neutrality. The correct amount of aeration may be the determining factor in some cases. In others the opposite may be true. In some plants in fact sulphuric acid has been added to neutralize a part of the alkalinity thus increasing the carbonic acid and lowering the pH.

4b. Rapid and thorough admixture of chemicals with the water reduces the dosage of chemicals required and promotes a better floc. Aeration may be one of the methods employed. The new water-softening plant at Neodesha, Kansas, employs aeration in the mixing tanks. Compressed air is applied through perforated brass pipes laid on the tank bottom (6). The recent development of the so-called "Aeromix" process is along this line.

4c. For many years aeration along with the wash-water was applied to filter beds as a good mechanical assistance. It is not generally so used today.

4d. Aeration in one form or another is generally employed in removal of iron and manganese. As previously stated it is necessary to supply oxygen in oxidizing ferrous bi-carbonate to ferric hydroxide and manganous bi-carbonate to hydrated manganese dioxide. At the same time it is an economical method of reducing the carbonic acid set free. The latter is of course responsible for the solubility of the iron or manganese in the lower state of oxidation. It is not present in the molecule in the higher state of oxidation. The amount of oxygen required is very small, 0.14 p.p.m. oxygen for 1 p.p.m. iron and 0.29 p.p.m. oxygen for 1 p.p.m. manganese. Weston states,

however, that it is frequently necessary to limit the aeration. This was found to be true at Reading and Middleboro, Mass., and at Superior, Wisconsin, aeration was abandoned. At Lowell, Mass., if aeration was carried beyond 50 percent of saturation with oxygen, manganese penetrated to and clogged the lower portions of the sand beds (7).

However, during the past year, experience with a small experimental plant for removal of iron from a well water in Flushing, New York, demonstrated that neutralization of free carbonic acid by lime hydrate completely precipitated the iron and completely removed it in a pressure filter in entire absence of oxygen and without any aeration. Determinations for dissolved oxygen both before and after filtration gave 0.00 p.p.m. oxygen, yet the iron was reduced from 4.80 to 0.10 p.p.m. Probably it was removed as neutral ferrous carbonate, $FeCO_3$, which is very insoluble in water, much more so than ferrous hydrate, FeO_2H_2.

Also during the last three years iron has been removed by aeration under pressure, Elgin process. By means of a relief valve and float-arrangement an air space under 65 pounds pressure is maintained in the upper portion of the space above the sand in the pressure filter and a certain depth of water maintained above the sand in the lower portion. The entering water is sprayed through the air. Such a plant, 200,000 gallons capacity, has been in operation for more than two years at Burton, Ohio, installed by the Elgin Softener Corporation, Elgin, Illinois. Filtration is at the rate of 3 gallons per square foot per minute. The raw water iron content has ranged from 2.5 to 8.0 p.p.m. and the filtered iron has rarely exceeded 0.10 p.p.m. At the State Reformatory for Women, Dwight, Illinois, a similar plant reduces the iron from 3.5 p.p.m. to 0.10 p.p.m. The capacity of this plant is approximately 250,000 gallons per day. (Communication from the Elgin Company, April 8, 1932, though tests have also been made by the Ohio State Board of Health.)

4e. Prevention of red water is accomplished by neutralization of free carbonic acid, since the latter is the carrier of iron to the tap. Aeration assists by reduction of the amount of carbonic acid to be neutralized. Effluent aerators to lower aggressiveness of the water are exemplified by plants at Albany, New York, Providence, Rhode Island, West Palm Beach, Florida, and Poughkeepsie, New York, (Donaldson (14) and Pirnie (8) (9)). We purposely use the term

"red water" because corrosion is not stopped nor appreciably reduced with a soft water.

4f. During the Summer season, the great body of water in reservoirs below 15–30 feet stagnates. Microscopic organisms as they die and other organic matter settle to the quiescent water. Resulting decay reduces oxygen to zero and proportionately increases carbon dioxide in accordance with the chemical equation for combustion, $C + O_2 = CO_2$. At the same time musty odors may develope. The most pronounced results are just below the thermocline and at the

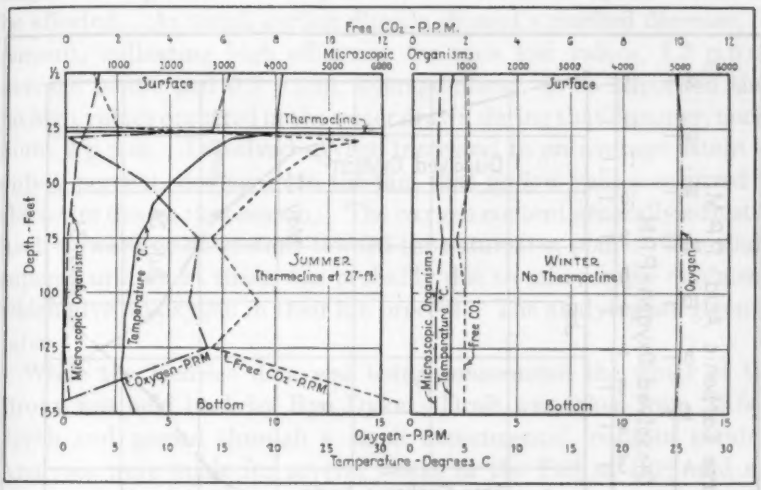

FIG. 1. QUALITY OF WATER AT VARIOUS DEPTHS—KENSICO RESERVOIR

bottom, with intermediate results between. It is frequently desirable to draw water from these deeper areas, since it is cooler, clearer, and more free from microscopic organisms. Aeration is a great help in permitting draft from greater depths. It removes the excess carbon dioxide and musty odors and adds air, thus rendering the water more palatable. The aerators of the Catskill system, one at Ashokan and one at Kensico, are each 500 feet by 250 feet, have approximately 1600 nozzles each, and spray the water 15–30 feet into the air. The Kensico aerator is used the year round and the Ashokan aerator similarly with the exception of the Winter. Details of these aerators are given on page 193 of the Manual of Water Works Practice (3) (15).

On July 13, 1917, the Kensico aerator was tested with the following results:

	BEFORE AERATION	AFTER AERATION	CHANGE
Temperature, °F.	58.0	58.5	0.5+
Free CO_2, p.p.m.	2.0	0.7	1.4−
Dissolved Oxygen, p.p.m.	9.35	9.55	0.2+

The water was drawn from 35–50 feet depth. The free carbon dioxide although small in amount was reduced by 65 percent.

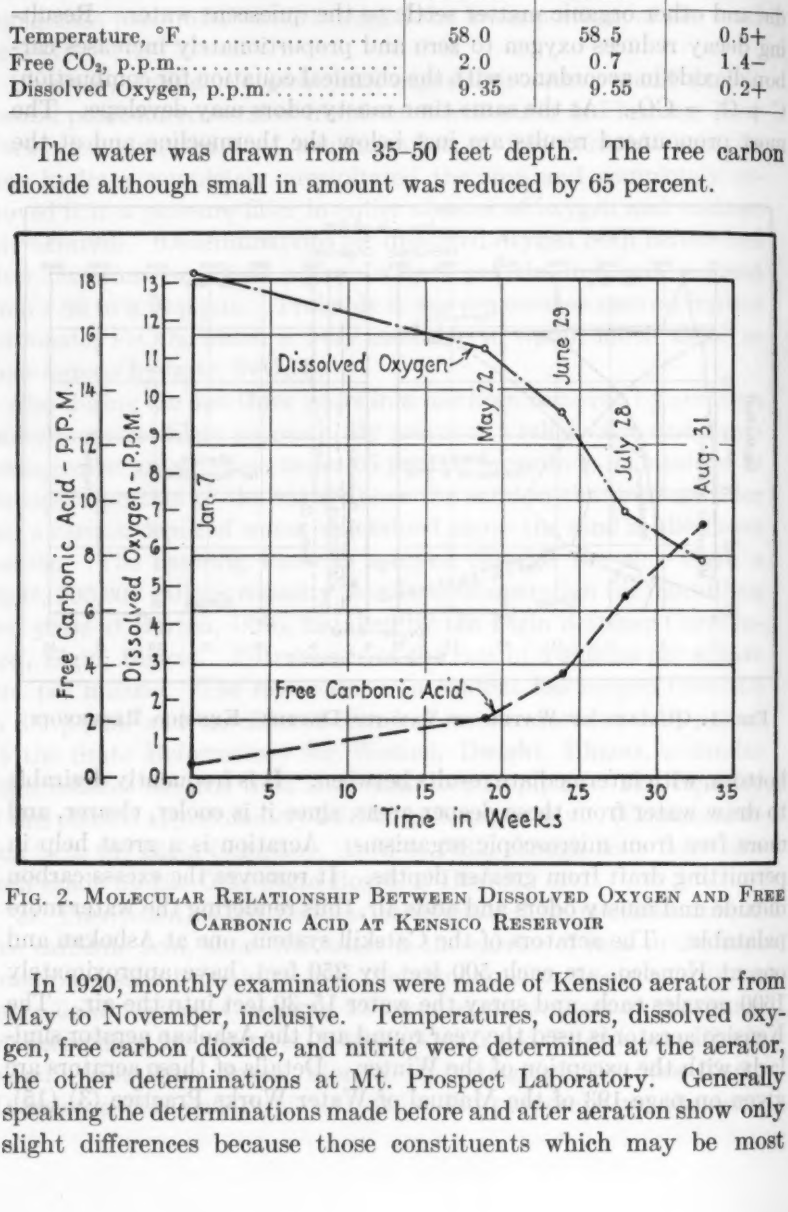

FIG. 2. MOLECULAR RELATIONSHIP BETWEEN DISSOLVED OXYGEN AND FREE CARBONIC ACID AT KENSICO RESERVOIR

In 1920, monthly examinations were made of Kensico aerator from May to November, inclusive. Temperatures, odors, dissolved oxygen, free carbon dioxide, and nitrite were determined at the aerator, the other determinations at Mt. Prospect Laboratory. Generally speaking the determinations made before and after aeration show only slight differences because those constituents which may be most

affected were present in such quantities that little change could take place. The odor following aeration in the May sample was due to the fact that the reservoir had just been treated with copper sulphate for Asterionella and the essential oil set free by the agitation caused the fishy odor. The total count for microscopic organisms was not appreciably affected. The orthotolidin test was probably due to manganese dioxide and averaged the same before and after. Determinations for nitrite were made in case it should be present in any quantity in the deep water and to learn what effect aeration might have upon it. It was present, however, in too small amount to be affected. As usual, carbon dioxide showed a marked decrease, 59 percent, indicating high efficiency for such low values, 1.2 p.p.m. average before and 0.5 p.p.m. average after. It so happened that no high values occurred in the water drawn during that Summer, maximum 2 p.p.m. Dissolved oxygen increased to an average extent of only 2 percent, due again to the fact that no low values occurred in the water drawn that season. The oxygen content generally adjusted itself upward or downward toward the saturation point. The slight supersaturation at times was probably due to microscopic organisms which give up oxygen in their life process. The analyses are given in table 1.

While the Kensico dam was being constructed the water of the Bronx was held back by Rye Dyke. Draft was taken from 25 feet depth and passed through a small experimental fountain aerator. Analyses were made for several weeks in the Fall of 1913 and are given in table 2. There was consistently a slight reduction in odor (lv), microscopic organisms (25 percent), and carbon dioxide (44 percent), and an increase in dissolved oxygen (7 percent), and temperature (1.1°C. or 2°F.). The nozzles were of the same design as later used in the larger aerators.

4g. Aeration has helped to break up colonies of fragile microscopic organisms,—in our experience: Uroglena, Synura, Asterionella, Anabaena, and Dinobryon. It is not wise to place too much reliance on this fact, however, as the individual unit may survive and cause trouble. Similar effect is sometimes caused by a heavy rain.

4h. Aeration assists in softening water by reduction of carbon dioxide and thus the amount of lime or other alkali required to neutralize it.

4i. Similarly aeration by removing carbon dioxide reduces the amount of lime needed in the lime and iron sulphate treatment of water.

TABLE 1

Investigation of Kensico aerator—1920

DATE	SAMPLING POINT	TEMPERATURE	TURBIDITY	COLOR	ODOR	NITRITE	DISSOLVED OXYGEN		FREE CO_2	ORTHOTOLIDIN TEST	TOTAL MICRO-ORGANISMS	AMORPHOUS MATTER
							Saturation	Determination				
		°F.	p.p.m.	p.p.m.		p.p.m. N	p.p.m.	p.p.m.	p.p.m.	p.p.m.	units per cc.	units per cc.
May 17	Before aeration	49	3	6	1a	0.002	11.5	12.7	0.2	0.10	685	625
	After aeration	50	2	7	2f	0.002	11.3	11.6	0.2	0.11	685	550
June 28	Before aeration	62	2	10	1v	0.001	9.8	10.3	1.1	0.03	585	345
	After aeration	62	2	10	1v	0.000	9.8	10.0	0.0	0.05	615	750
July 19	Before aeration	65	2	7	1v	0.001	9.7	9.7	1.3	0.07	125	1,250
	After aeration	65	2	7	1v	0.001	9.7	10.1	0.7	0.07	120	1,000
Aug. 24	Before aeration	65	2	6	1v	0.000	9.7	10.3	1.5	0.16	185	1,350
	After aeration	65	2	6	1v	0.000	9.7	10.9	0.7	0.14	90	1,300
Sept. 27	Before aeration	65	2	8	1v	0.001	9.7	11.7	1.5	0.14	805	1,575
	After aeration	65	2	8	1v	0.001	9.7	11.9	0.7	0.12	745	1,200
Oct. 25	Before aeration	60	2	6	1v	0.000	10.0	9.5	2.0	0.16	150	850
	After aeration	61	2	7	1v	0.000	9.9	9.9	0.7	0.14	120	850
Nov. 22	Before aeration	48	2	4	1v	0.000	11.6	10.6	0.7	0.11	135	1,050
	After aeration	48	2	6	1v	0.000	11.6	11.7	0.2	0.14	125	850
Average	Before aeration	59	2	7−	1v	0.001	10.3	10.7	1.2	0.11	380	1,005
	After aeration	59	2	7+	1v	0.001	10.2	10.9	0.5	0.11	355	930

Important genera were Asterionella, Synedra and Tabellaria.

TABLE 2

Investigation of Rye Dyke aerator—1913

Important genera were Asterionella and Tabellaria.

TABLE 2

Investigation of Rye Dyke aerator—1913

DATE	SAMPLING POINT	TEMPERATURE	ODOR	OXYGEN	FREE CO_2	TOTAL MICRO-ORGANISMS	REDUCTION IN			INCREASE IN	
							Odor	Free CO_2	Micro-organisms	Temperature	Oxygen
		°C.		p.p.m.	p.p.m.	units		p.p.m.	units	°C.	p.p.m.
Sept. 25	Before aeration	18.0	2v+e	10.1	4.0	375					
	After aeration	19.0	1v	10.8	1.3	130	1v+e	2.7	245	1.0	0.7
Oct. 2	Before aeration	20.0	2v	11.1	9.3	495					
	After aeration	21.5	1v	11.8	8.5	205	1v	0.8	290	1.5	0.7
Oct. 17	Before aeration	14.5	2v	8.8	2.6	355					
	After aeration	16.0	1v	8.9	1.9	250	1v	0.7	105	1.5	0.1
Oct. 23	Before aeration	13.0	2v	10.6	2.6	385					
	After aeration	13.0	1v	11.7	1.7	130	1v	0.9	255	0.0	1.1
Oct. 30	Before aeration	13.0	2v+m	11.3	5.3	285					
	After aeration	14.0	1v	13.3	2.2	260	1v+m	3.1	15	1.0	2.0
Nov. 6	Before aeration	10.0	2v	10.2	4.0	180					
	After aeration	10.5	1v	11.0	2.2	235	1v	1.8	55+	0.5	0.8
Nov. 14	Before aeration	10.0	1v	10.5	4.4	465					
	After aeration	11.0	1v	11.3	1.8	305	0	2.6	160	1.0	0.8
Nov. 20	Before aeration	8.0	1v	10.9	4.4	950					
	After aeration	10.0	1v	10.3	1.8	960	0	2.6	10+	2.0	0.6−
Nov. 28	Before aeration	6.5	1v	11.0	4.8	385					
	After aeration	8.0	1v	11.4	2.6	210	0	2.2	175	1.5	0.4
Dec. 4	Before aeration	7.0	1v	10.6	4.0	280					
	After aeration	8.0	1v	11.5	1.8	455	0	2.2	175+	1.0	0.9
Average	Before aeration	12.0	2v	10.5	4.5	415					
	After aeration	13.1	1v	11.2	2.6	315	1v	2.0	100	1.1	0.7

Important genera were Asterionella, Melosira, Synedra and Aphanizomenon.

4j. The place of aeration in a plant whether of the raw water, of the coagulated water, or of the filtered water depends upon the purpose and the particular water to be handled. Double aeration is employed in some plants. Pirnie (8) (9) mentions West Palm Beach, Florida, Providence, Rhode Island, and Danville, Virginia. Greenwich, Conn., aerates the raw water.

4k. Aerators or fountains appeal to the popular imagination and add to the beauty of water works. The popular mind pictures the aerator as purifying the water. The psychological effect is good.

5th. Methods: The theory and practice of aeration has been recently interestingly described by Prof. Langelier. Early work on the mechanism of air absorption in relation to pollution in New York Harbor was published by Phelps in a series of papers dated from 1911 and later work in collaboration with Streeter in connection with reaeration of river water was published in 1925 (11) (12). Several other references are given in Prof. Langelier's paper.

The entire subject of aeration is very well covered in "The Microscopy of Drinking Water" revised by Fair and Whipple (16), 1927, pages 408–422. On pages 411 and 412 are four tables showing the rapid reduction of carbon dioxide, hydrogen sulphide and odor in experiments in which water was allowed to pass through the air in drops. On page 421 different types of nozzles in common use are pictured. On pages 415, 418 and 419 are several illustrations of various forms of areation, riffles, fountains, etc.

5a. Simple exposure to air as in passing through a basin or flowing through an aqueduct has an aerating effect. Several years ago we made a study of the Croton supply at the time the policy of drawing water from a depth was started. Samples were taken as the water left Croton Lake and at intervals to the City. Starting with high free carbon dioxide and reduced oxygen the water gained oxygen and lost carbon dioxide as it flowed to the City. Time was the chief factor.

The Metropolitan Sewerage Commission of New York carried out experiments on rate of absorption of oxygen from air under quiet conditions, using both fresh water and salt water from which the air had been exhausted. The rate of absorption was rapid at first, decreasing as the degree of saturation increased. Thus 10 percent of the saturation value was taken up in the first hour, over 50 percent in the first day. The rate decreased to less than 1 percent per 24 hours requiring 7 to 10 days for saturation (13).

5b and 5c. Water pouring over weirs or in cascades down steps is not as effectively aerated as by spraying. Tests made by us in iron removal have shown slow drop in free carbon dioxide. At Rockaway Park pumping station high iron well water allowed to drop from a riser pipe onto an apron and drop again to a sedimentation basin lost only 10 percent free carbon dioxide (40 to 36 p.p.m.) at filtration rate of 50 mgad. and 8 percent (46 to 42 p.p.m.) at 125 mgad. At the time the raw water contained 16–17 p.p.m. iron. Numerous steps are required. After addition of seven more short drops in steps, nine splashes in all, carbon dioxide was reduced 25 percent (55 to 41 p.p.m.) at half filtration rate, (69 mgad.) and 18 percent (49 to 40 p.p.m.) at full rate, (114 mgad.). The iron at this time in the raw water was 5–6 p.p.m. (14).

Whipple (16) states that the effluent from the filters at Cambridge, Mass. passing over riffle plates loses 20–30 per cent of carbon dioxide and aromatic odor, also that at Portsmouth, Va., perforated-pan aerators reduced carbon dioxide 30–40 percent, the holes being $\frac{1}{4}$-inch in diameter and the fall 8 feet, that experiments at Lawrence, Mass. showed that water devoid of oxygen could be saturated by falling $3\frac{1}{2}$-feet in drops.

5d and 5e. Trickling devices are probably more effective but sprays are most effective as indicated by high percentage removal of carbon dioxide, previously quoted in this paper as averaging 70 percent.

At Memphis, Tenn., the air-lift used to raise the water from great depths reduces the free carbonic acid from 110–120 p.p.m. to 35–40 p.p.m., 75 percent removal, and the coke aerators reduce the free carbonic acid from 35–40 p.p.m. to approximately 4 p.p.m., 90 percent removal.

5f and 5g. Blowing air through water by pressure by means of perforated pipe or porous plates, or drawing air through as by the Aeromix process doubtless are effective although efficiency as to absorption increases with the fineness of the bubbles. Even such a readily absorbed gas as chlorine requires the fineness of a porous plate to be effective. If the small bubbles coalesce into large bubbles they shoot to the top of water and break on the surface or absorb downward at the surface. It is necessary to trap chlorine gas by concrete aprons above the water to obtain full efficiency. It is probable that oxygen would not absorb as rapidly as chlorine. The efficiency of the streaming air to remove carbon dioxide is another problem. Ex-

perimental aeration by drawing air through at Mt. Prospect Laboratory has never demonstrated much efficiency nor success in connection with iron removal. Data as to absorption of carbon dioxide is available at those plants where recarbonation is practiced.

5h. The latest method that has come to our attention is the Elgin process in which water is sprayed into air under 65 pounds pressure. Ferrous iron is readily oxidized with loss of carbon dioxide and also hydrogen sulphide if present. This process has been previously mentioned under *4d.* It is the opposite of the deaerating method in which water is sprayed into a vacuum to remove oxygen in control of corrosion in boilers, which has been known for several years to be very effective.

It is not our intention to go into engineering details as to equipment such as best form of spray-nozzles, head required, etc. We must apologize that due to lack of time we have not made a comprehensive search of the literature concerning aeration. The above, however, should offer suggestions for discussion.

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DISCUSSION

W. F. LANGELIER:[2] It is the writer's belief that, generally speaking, water-works men are inclined to rate the benefits to be derived from aeration, either too highly or too lightly. Dr. Hale has discussed this technical and somewhat controversial subject efficiently and without prejudice. Much of the existing journal literature on the subject of aeration contains data which are not entirely dependable and from which it is not safe to draw too general conclusions. I believe there is a need for the accumulation of additional accurate performance data and for studies of the relative merits of "spray" versus "bubble" aeration for the several functions wherein aeration is indicated.

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FLOC FORMATION AND MIXING BASIN PRACTICE

By George E. Willcomb[1]

The purpose and scope of this paper is to bring out the salient points of floc formation and the application of these features to mixing basin practice by means of the following topical headings:

(1) Conditioning a preferable term to mixing.
(2) Factors affecting the composition of floc.
(3) Factors influencing the formation of floc.
(4) Types of agitators used in plant practice.
(5) Modern mixing or conditioning practice.

CONDITIONING A PREFERABLE TERM TO MIXING

For years. ever since coagulation of water by a chemical was instituted, the term mixing has been used in a totally wrong sense of the word to describe the process of accretion or agglomeration of the particles of hydrate flocs into masses of sufficient weight and bulk so that they would sink to the bottom of the clarifying basins when the treated water was introduced into them. In other words, mixing is the wrong word to ascribe to the process of preparing floc for the clarifying action provided in the retention period of the tank process to follow.

By mixing, is meant the rapid diffusion of the chemical coagulating solutions throughout the body of water being treated and, due to the fact that floc formation is almost instantaneous, mixing may well include the idea of floc formation; but the coalescing of the floc particles into masses is a distinct operation involving time and physical change of the colloidal flocs formed in the mixing action into new and heavier forms. This process is best described by the word conditioning and it is therefore suggested that the word conditioning be substituted for mixing in describing purification plant layouts. It is interesting to note that Fleming (1) in discussing the coagulation of waters of high turbidity sharply differentiates between mixing and conditioning.

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While the subject of misnomers is being discussed it might be well to take up other glaring examples of misapplication of descriptive terms. Take for instance the term settling basins. This term from the beginning was used to describe the basins used in the slow-sand process to settle out the suspended solids without the use of chemical coagulants. It would seem wise to retain this term to describe preliminary processes of treatment, such as grit-chambers and the like, for preparing turbid waters for subsequent chemical treatment. In a like manner coagulating and clarifying basins used indiscriminately are confusing. The complication of modern purification plant design demands concise and descriptive language in articles purporting to accurately describe the intricacies of the process. It might therefore be well to ascribe to the whole process of floc formation and conditioning the term coagulation and limit the term clarification to the function of the retention tanks following the mixing and conditioning processes in which the flocs are deposited by gravity.

To simplify what has been written above, the following arrangement of terms used in modern purification practice suggest themselves as being concise and adequately descriptive:

(1) Settling action. Preliminary treatment comprising grit-chamber and plain sedimentation tanks not using alum.

(2) Coagulation. (a) Premixing and mixing devices operated mechanically or by gravity flow. (b) Conditioning-gravity flow and mechanically agitated tanks, air agitating and aerating devices.

(3) Clarification. The deposition of the floc formed by the conditioning process in tanks of sufficient size such that the tank effluents are suitably prepared for filtration purposes. Tanks of the Dorr type with mechanical means of continuous evacuation of the deposited sludge come under this heading.

FACTORS AFFECTING THE COMPOSITION OF FLOC

To the early operators of mechanical filters, from 1897 to 1900, the composition of hydrate floc was looked upon as being a simple thing. It was regarded as being composed of aluminum hydrate $Al(OH)_3$ and was believed to be always formed when aluminum sulphate was added to water containing sufficient alkalinity to react with the quantity of alum added, leaving about 10 p.p.m. of alkalinity in excess. The theoretical reduction in alkalinity for the number

of grains of alum added was always worked out and when the alkalinity reduction was more or less than the theoretical, a great deal of research was undertaken to ascertain why these discrepancies occurred. This research finally led up to the epoch making work of Clark, Theriault and Miller of the United States Hygienic Laboratories in 1923–25. Hatfield (2) gives a concise summing up of these papers in his "Alum Floc, What It Is." It is brought out in this paper that the point of most rapid floc formation is quite definitely controlled by the pH of the solution, and that there is a broad zone of pH within which the effluent from the filters contain no dissolved alumina. It has been demonstrated that the optimum reaction pH point varies for different waters, and that the same is true for the broad zone of insolubility of aluminum. General plant practice has demonstrated that each particular water possesses an alum reaction characteristic to itself and that there are factors other than the reaction pH to be considered.

The work of Clark, Theriault and Miller demonstrated that working with weak solutions of pure salts the optimum pH value for floc formation centred in a narrow zone in the vicinity of 5.50. It was also demonstrated that from pH values of 4.0 to 5.5 the floc is a basic aluminum sulphate of an approximate formula of $5Al_2O_3$—$3SO_3$— whose sulphate content steadily decreases with increasing alkalinity of the reacting solution until at pH 8.5 there is no sulphate and the floc consists entirely of aluminum hydrate $Al(OH)_3$. At a low pH favoring the formation of complex basic sulphate ions these sulphate ions may be replaced by other negatively charged polyvalent ions and this suggests that the mechanism of color removal may be due to the formation of a color floc produced by the interchange of sulphate ions with the negative polyvalent ions composing the coloring matter. This is substantiated in a practical way by Norcom (3) who in his work with highly colored waters has found a critical reacting point for color removal in the vicinity of pH 4.4. The work of these investigators may be summed up as follows: when a polyvalent sulphate solution is added to a water two kinds of floc may be formed depending upon the resultant pH of the solution. From pH 4.0 up to 5.0 the strong coagulating power of the tri-valent aluminum ion acting on the negatively charged colloidal color is in evidence, forming what is called color floc. The second kind of floc known as alum floc approaches completion of precipitation at pH 5.4 and is complete at pH 8.5. It is therefore between pH 5.4 and 8.5 that the

best clarification of turbid waters occur which are dependent upon the formation of an alum floc consisting of hydrate ions. It is thus seen that the successful clarification of a water depends upon three factors: there must be added a certain minimum quantity of aluminum ions, there must be present an anion of strong coagulating power such as sulphate, and the hydrogen ion concentration must be properly adjusted for the work to be done, i.e., color or turbidity removal.

The above statements apply to aluminum flocs only. In the case of flocs formed by ferric chloride, ferrous and ferric sulphates, the conclusions are the same as for aluminum except that ferric flocs do not tend to dissolve at higher pH values and therefore require less careful adjustmet of the pH. In addition, ferric flocs begin to form at a somewhat lower pH value than do alum flocs. Iron floc being heavier than alum tends to sink sooner in clarifying tanks and hence this coagulating agency is favored when difficulty is experienced in sinking the floc or in cases where the clarifying tanks are of too short a retention period. Iron floc has a lower color removal efficiency, hence when colored waters are being treated alum coagulation is favored.

FACTORS INFLUENCING FORMATION OF FLOC

The previous topic has described in brief conditions affecting the composition of floc, the present topic concerns itself with factors influencing the formation of a floc of appreciable size from the colloid floc formed just after the coagulating chemicals are added. As has been stated above, due to the high dilution offered by the water treated by the chemicals, the reactions are almost instantaneous and a colloid floc is formed at once which is almost undiscernible by the naked eye. The problem involved is to increase the size of the floc particles by accretion and this is best brought about by beating them together. Every operator will remember his experience in the school laboratory in the gravimetric precipitation of barium sulphate by means of adding the reagent barium chloride. He will recall that in order to produce a precipitate that would not go through the pores of the filter paper he had to stir the solution violently while the reagent was being added. In some stubborn cases he had to mascerate filter paper in the solution to obtain a clear filtrate. It is little wonder then that filter operators from the very beginning resorted to agitating the colloidal alum floc to obtain a settleable floc.

It was not long before stirring machines were in use on the laboratory benches of all rapid treatment plants and the operators were ascertaining the best mixing and conditioning practice for their particular type of water in an empirical way. The unfortunate thing was that none of the early rapid plants were provided with agitating devices for the production of a conditioned floc. Rapid plants from 1897 to 1909 generally were subject to ill conditioned water at certain times of the year when the fine floc would pass from the clarifying basins onto and through the filters producing a turbid effluent. It was not until 1909 that an adequate conditioning basin was built at New Orleans.

The best way of determining the proper time of mixing and conditioning in the laboratory is to make use of some mechanically operated stirring apparatus. These may be bought at chemical supply houses at a cost of from $200.00 for the electrically operated and heated devices to $20.00 for the hand operated type. They are best obtained so as to agitate six samples at one time. The power may be either electrical, water, or manual. Water motors are to be preferred as they do not involve complicated and expensive control apparatus. Where electric drives are used a variable speed motor and reducing pullies may be employed or some form of friction disc drive may be used that gives a great variety of impellor speeds. One of the best stirrers thus far developed is a home-made affair making use of a second-hand motor, reduction gears and a horizontal shaft working on the rocker-arm principle that gives an up and down or butter-churn action to the stirrer. This device uses six one quart cylindrical pickle jars and the dashers consist of flat metal discs fastened to the vertical rods attached to the horizontal rockerarm. The great advantage of this up and down motion of the dashers is, that the motion imparted to the flocs in each of the six samples is exactly the same so that an accurate deduction may be made as to just what alum dose gives the quicker floc and settling action. Instances have been observed with vertical rotating shafts where the worn thrust bearing has caused such a decided eccentricity as to give better floc formation with a 1.5 grain dose than with a 2.0 grain dose of alum in an ascending coagulating series, due to the more violent agitation brought about by the varying eccentricities of the different shafts.

As regard the type of agitating shaft used, the writer prefers a rapidly driven straight glass rod with no extensions or enlargements at the bottom, driven by a water motor, providing, of course, that

suitable bearings assure non-eccentricity of shaft alignment. Such an agitator can be used at a high shaft speed bringing about two motions within the liquid medium: a rotary and an up and down motion of the particles close to the shaft. This prevents the floc travelling along at the same speed as the water currents. In all experiments the agitation in the bench set-ups should duplicate in as far as possible the actual conditions obtaining in the plant conditioning process, such that the dose worked out empirically will give like results in actual plant operation. If such is not the case then the bench apparatus is poorly designed and should be modified accordingly.

Generally speaking, the lower the temperature of the water the more retarded the chemical reaction and consequent floc formation. Due to the increased viscosity of the water and changed surface tension of the flocs themselves, the particles resist coalescing tendencies and consequently require more prolonged agitation to increase the size of the flocs such that they will settle out.

In the case of clear waters of moderate color, increasing the turbidity by artificial means has been productive of improved conditioning to the extent that the conditioned floc has been made to sink in a small sized clarification basin that otherwise would have gone over onto the filters in the form of a colloid. Clay may be added to the extent of producing an artificial turbidity of from 20 to 50 p.p.m., or coagulated sludge from the basins may be pumped into the raw water suction inlets. The writer has obtained excellent results with the use of kaolin which is a clay naturally high in alumina.

MEASUREMENT OF FLOC

In order to know what is taking place in the different compartments of clarifying and conditioning tanks it would be very helpful if there was an easy and accurate method of quickly determining the amount of hydrate floc deposited in these compartments. Then, too, it would be very interesting to know the actual amount of floc passing through in the effluent of a rapid filter at different periods of a filter run. To make a gravimetric determination of alumina is a tedious and time consuming operation and is out of the question for routine work.

Three methods for the rapid routine estimation of floc have been proposed and are in practical use. These methods are: The Dissolved Color Method of Nasmith (4), The Floc Detector of Baylis (5),

and the Superimposed Cover Glass Slide Method developed by the writer.

The dissolved color method was worked out by Nasmith during the operation of the experimental filter plant for the City of Ottawa, Canada. I t is based on the fact that if a colored water containing alum floc is treated with dilute sulphuric acid, the coloring matter adsorbed by the floc would go into solution and this color could be measured accurately in the usual way by comparing with color standard tubes. It was also argued that the quantity of color so found could be fairly assumed to represent a similar quantity of floc. The method consists of acidifying a sample of the raw water which should preferably have a color of at least 30 p.p.m. The color of this acidified and filtered sample is obtained likewise the color of the plain filtered raw water, which should be higher than the acidified water. A series of 50 cc. Nessler tubes are made up as standards containing colors of 5, 10, 15, and 20 p.p.m. made by diluting aliquot portions of the acidified, filtered raw water with distilled water.

Now supposing the raw water filtered had a color of 30 p.p.m. and this filtered water acidified with five drops of dilute (1:4) sulphuric acid had a color of 20 p.p.m. Suppose a sample of the raw water treated with alum were to be collected immediately after the addition of the alum. If this coagulated water were to be acidified and filtered the color of the resulting filtrate should be 20 p.p.m. If a sample from the outlet of the clarifying basins was collected, acidified and filtered, and gave a color of 5 p.p.m. then form its ratio to 20, the percentage reduction in the amount of floc deposited is found to be 75. This method is practical and has been used by the writer in the routine determination of floc removal efficiency of clarifying basins.

The Floc Detector of Baylis consists of making a standard suspension of alum floc in a litre flask prepared by actually adding an aliquot dose of alum to a sample of tap water. From this a series of diluted litre standards are prepared having floc turbidities of 0.1, 0.05, and 0.02 p.p.m. etc. The value of the method is, that one is comparing turbidities with standards made of actual hydrate floc. The standard fiask and that containing the unknown are placed in an illuminated box and the turbidities compared. It is understood that this method is used as routine at the Baltimore Filters.

The writer's method as developed at Albany takes into account the clarity of the water in general, whether composed of floc or other suspended matter, although in the case of a basin or filter effluent the

turbidity is always due to colloidal or suspended hydrate. The method is used in the routine determination of the turbidity of clarification basin and rapid filter effluents. The apparatus used consists of a ground-glass background, 12 inches wide and 24 inches long. Black bands one quarter and one half inch wide are painted along the long axis of the glass background. The glass is set up on a shelf in front of a window. The vessels to hold the standard and the unknown consist of square battery jars, 4 inches by 4 inches by 8 inches high (Arthur H. Thomas, Catalog No. 2098). These jars are of extra clear glass and offer little distortion of the images of the black lines as viewed through the water. The turbidity standards consist of clusters of microscopic cover-glass slides superimposed one upon the other to impart a grayness to the distilled water in the jar when the slides are looked at through the jarful of water in a horizontal direction. One slide looks slightly gray through the water, ten slides look decidedly gray. The procedure is to fill the left-hand vessel with distilled water and the right-hand one with the unknown water. Then looking horizontally through each jar at the black lines on the ground-glass background, clusters of slides of varying number are held behind the distilled water until the grayness of the slide cluster matches the turbidity of the unknown. In practice clusters of slides from one to ten in number are held together by clamps for convenience.

Turbidities of from 0.05 to 1.00 p.p.m. are accurately determined in this way and the value of the method lies in the fact that once the clusters of slides are standardized against known turbidity standards prepared in the customary way the turbidity value of the slides remains constant once for all, and does not have to be continually restandardized as do silica turbidity standards. In other words the chief difficulty encountered in the use of silica turbidity standards, the error of progressive lessening of turbidity values due to agglomeration of the silica particles on shaking, is absolutely done away with. Table 1 gives standardized turbidity values for clusters of cover-glass slides $\frac{1}{16}$ inch thick.

EFFECTS OF CONDITIONING ON VARIOUS TYPES OF WATER

Turbid waters

The experience of various investigators with different kinds of water will now be summarized. Technical literature abounds with the results of experimentation using various forms of agitation veloci-

ties and duration of conditioning. To simplify the summarization the discussion will be confined to the results accomplished on conditioning three types of water: highly turbid, soft colored, and low turbidity waters grossly contaminated by sewage and trade wastes.

Conditioning of highly turbid waters has naturally predominated in the middle west and south. The following is a summary of Hoover's (6) work at Columbus, Ohio. With actual plant results it was found that 75 minute conditioning at a 0.3 foot per second velocity was inadequate for waters of from 300 to 600 p.p.m. turbidities. Bench experiments were made to determine optimum velocities and

TABLE 1

NUMBER OF SLIDES	TURBIDITY	NUMBER OF SLIDES	TURBIDITY
1	0.05	7	0.55
2	0.10	8	0.60
3	0.20	9	0.70
4	0.30	10	0.80
5	0.40	11	0.90
6	0.50	12	1.00

TABLE 2

VELOCITY (FEET PER SECOND)	TIME OF CONDITIONING	TURBIDITY OF SUPERNATANT LIQUID
Vigorous	30 seconds	
0.13	30 minutes	90
0.22	30 minutes	30
0.60	30 minutes	2

retention times of conditioning. The waters were agitated in glass jars, allowed to settle 30 minutes and the turbidity of supernatant liquid obtained. The results of treating a water of 325 p.p.m. turbidity with 4.0 grains of alum with various agitation velocities shown in table 2.

It is believed to obtain like results with quick mixing followed by conditioning at low velocities, at least 6.0 to 8.0 grains of alum would have been required. It was stated that in water softening practice 87 to 90 percent of the reaction is effected by proper conditioning.

Langelier (7) in his experiments with conditioning in connection with the design of the Sacramento, California, plant did much experimentation with turbid waters, employing varying velocities and

retention periods, and ascertaining short circuiting conditions within various types of tanks, especially mechanically agitated circular tanks. The results of his experiments may be summarized as follows: Using alum doses of from 0.5 to 0.8 grain per gallon, velocities of from 0.2 to 2.0 feet per second, and retention periods of from 10 to 20 minutes, it was found that turbidities up to 200 p.p.m. could be handled so as to produce a well conditioned water for subsequent clarification. It was also found that prolonged retention at velocities up to 2.0 feet per second and over did not cause a breaking up of the floc; that, in general, the highest velocities produced the most rapid flocculation, and that other conditions remaining constant, an increased alkalinity necessitated either a more prolonged period of agitation or an increased dosage of alum.

Cox (8) in an extended experimental study of the efficiency of conditioning under Reading, Pa., conditions drew some interesting conclusions as to the inter-relation between retention period and velocity. He concludes that agitation may be analyzed into intensity and time factors which are roughly inversely proportional to each other. That for the conditions obtaining at Reading, the following range in conditioning is possible: 20 minutes at a velocity of 0.25 feet per second and 8 minutes at a 0.8 feet per second rate. He concludes that the importance of thorough conditioning would seem to apply to the coagulation of colored waters and to water softening to a greater extent than to the coagulation of the average turbid water.

The general conclusion to be drawn from the experimentation in conditioning highly turbid waters, is that an appreciable amount of alum may be saved by conditioning the water from 20 to 30 minutes at velocities of from 0.5 to 1.0 foot per second. Some waters require violent agitation at the beginning followed by a decreasing velocity at the end. Care should always be taken to keep the floc in suspension and to design the inlets to the clarification tanks large enough so that the floc will not be broken up. With these precautions a well conditioned floc is produced capable of settling out in a minimum time in a suitably designed clarification tank.

Soft colored waters

From the results obtained by Norcom (3) and Saville (8) color removal is due to two factors: the neutralization of the negative electrical charge carried by the color colloid by the positive charges of the aluminum and hydroxide ions; and secondly, the mechanical inclosure

of the colloid color by the hydrate floc. Should the water be difficult to clarify, it is generally due to the fact that the color colloid bears a positive electrical charge and the only removal is that brought about by the mechanical enmeshment by the aluminum hydrate.

The mechanical enclosure of the color colloids by the floc is hastened by adequate conditioning. Tests indicate that colored waters require more prolonged conditioning than do turbid waters. The velocity of conditioning is lower than that of turbid waters and especial care should be taken that the comparatively delicate color floc is not broken up in its passage through flumes and inlet ports. It has been found that irrespective of the conditioning, if the chemicals have not been carefully proportioned color removal is impaired.

Clear sewage contaminated waters

This type of water predominates in the east and the writer is connected with the operation of several purification plants treating waters of this type, one of which handles a water whose average yearly coli index is 49,000 per 100 cc., the average turbidity being only 9 p.p.m. Such waters are most difficult to coagulate and require large alum doses and optimum conditioning. The reaction pH during the initial mixing varies with the amount of oxygen consumed in the raw water. As the organic content increases the pH at which optimum coagulation occurs decreases in direct proportion. The effect of varying amounts of oxygen consumed on the amount of alum necessary to produce a well conditioned water from a typical highly contaminated raw water like the Hudson River is shown in table 3.

In all bench coagulation experiments the writer has made use of the terms in table 4 in an endeavor to express the progressive degree of coagulation by a number called the coagulation index of a particular water. Thus, as increasing doses of alum are added to a given water the corresponding degree of flocculation is indicated by an index number descriptive of the results obtained.

This scheme of having the degree of flocculation expressed by numbers enables one to plot the indices against other factors in making coagulation studies. The use of numerical indices is shown in table 5 which clearly shows the effect of retention period and velocity of agitation on the coagulation of clear contaminated waters.

It will be noted that waters of this type do not require violent agitation. If we assume a coagulation index of 5 as being the minimum for waters to be settled in two hour clarifying basins then it is evident

that with a 0.8 foot velocity a retention conditioning period of 10 minutes is necessary, and with a 0.4 feet velocity, 14 minutes conditioning is required. Table 5 forcibly illustrates the efficiency of

TABLE 3

OXYGEN CONSUMED, 30 MINUTE BOILING PERIOD (P.P.M.)	REACTION, pH	GRAINS OF ALUM PER GALLON NECESSARY FOR PROPER COAGULATION
10	6.70	1.5
11	6.70	1.5
12	6.55	2.1
13	6.50	2.8
14	6.50	2.9
15	6.45	3.1
16	6.44	3.2
17	6.42	3.5
18	6.41	3.8
19	6.40	4.0
20	6.39	4.1
21	6.38	4.5
22	6.35	4.8
23	6.34	4.9
24	6.32	5.0
25	6.31	5.1

TABLE 4

Method proposed of ascribing an index number to the progressive changes in floc formation brought about by varying alum doses or conditioning changes

INDEX NUMBER	CORRESPONDING DESCRIPTION OF FLOC
0	Colloidal floc, absolutely no signs of agglomeration
2	Faint. Floc in minute particles not discernible by the ordinary observer
4	Distinct. Floc well formed but uniformly distributed
6	Decided. Flocs large in size and about to precipitate
8	Excellent. Deposition of floc well established but not complete
10	Complete. Floc practically all deposited, supernatant liquid practically clear

conditioning for it will be noted that the 2.5 grain alum dose produces no floc at all when the agitation is omitted.

None of the plants under the writer's control built between 1900–11

were provided with mixing or conditioning devices. The gravity plants had their alum introduced just before the raw water meter, and the pumping plants had their alum dose added to the pump suction. All of these plants were provided with baffled 4 to 10 hour clarifying basins. The agglomeration of the floc occurred in the basins themselves and in the winter months the floc went through the basins as colloids and passed through the filters into the effluents. This is a common occurrence in waters high in sewage emulsoids and with no conditioning provided for.

TABLE 5

Effect of varying velocity and retention period on the degree of coagulation expressed by coagulation indices

Raw water: Turbidity, 6 p.p.m.; color, 20 p.p.m.; oxygen consumption, 12 p.p.m.; alum, 2.5 g.p.g.

NUMBER OF MINUTES AGITATION	COAGULATION INDEX		
	Velocity of agitation: Feet per second		
	0.0	0.4	0.8
2	0	0	0
4	0	0	0
6	0	1	2
8	0	2	4
10	0	3	5
12	0	4	6
14	0	5	7
16	0	6	8

The effects of conditioning clear polluted waters during periods of low temperature in actual plant practice are shown in table 6.

In table 6 Plant A is the new Helderberg Plant of the new Albany water supply treating the Hannacroix Creek system. This plant was designed by Whitman, Requardt and Smith in 1931 and although tuned up has as yet not been placed in operation. It is expected that this plant will be placed in commission during July, 1932. The design is a good example of what can be done with gravity flow conditioning when there is plenty of head available. The chemicals are mixed either by passing through an initial aerator or by passing through a flume and valve following aeration. The conditioning basins, three in number, are each provided with eight baffled passages,

the first passage being fitted with over and under baffles, and the remaining seven being of the round the end type. The three conditioning basins may be operated in series or in parallel. At a normal flow of 32 m.g.d. the combined basins will give a retention period of 20 minutes and a velocity of 0.9 foot per second when operating in series. As the water enters each basin successively it meets with one over and under passage. This tends to break up any stratification in flow induced by the round the end passages of the preceding basin. When operating in series the water may be taken off to the clarifying basins at the end of each compartment thus providing 7, 14, and 20 minute conditioning periods respectively. When operating in parallel the retention period is the same, namely 20 minutes, but the velocity of agitation is reduced from 0.9 to 0.3 foot per second. It is thus evident that this type of conditioning is most elastic and admirably adapts itself to seasonal changes in the raw impounded reservoir water.

TABLE 6

PLANT	CAPACITY (MILLION GALLONS PER DAY OUTPUT)	CONDITIONING			CLARIFYING BASINS (HOURS RETENTION)	ALUM AS Al_2O_3 (P.P.M. IN EFFLUENT DURING WINTER)
		TYPE	Velocity of agitation (feet per second)	Period of minimum agitation (minutes)		
A	40	Baffles	0.9	20	3	0.050
B	35	Gravity, spiral flow and compressed air	0.5	15	10	0.075
C	10	Baffles	0.75	1	2	1.500
D	2	Baffles and air	0.25	30	3	0.050
E	2	Gravity, spiral flow and aeration	0.25	5	96	1.700
F	1	None			1	2.000

Plant B of table 6 is the Hudson River Plant of the old Albany water supply system. This plant provides an initial high velocity mix by introducing the alum into the suction of the centrifugal pumps. This is followed by a six hundred foot passage through a conduit to a high head jet aerator. From the aerator tray the water drops vertically 30 feet and enters a spiral flow cylindrical tank of 15 minutes retention. The water enters this tank tangentially at the bottom and leaves at the top in like fashion.

In the operation of this conditioning system a most interesting phase has been developed, demonstrating without an element of doubt the essential qualities of adequate conditioning. The aerator, being exposed to the air, has always been taken out of commission in December to prevent its freezing. Following the taking off of the aerator the floc has always assumed colloidal properties that has resulted in the passage of the floc through the rapid sand filters onto the surface of the secondary slow-sand finishing filters. This colloidal turbidity has always resulted in shortening the runs of the slow-sand filters from a normal average of 30 days to an average of 10 days. This occurs at a time when the demand on the plant for water is at a maximum due to consumers wasting large amounts of water to prevent their taps from freezing.

TABLE 7

Turbidities expressed in parts per million

	AERATORS OFF—POOR CONDITIONING	WINTER AERATION IN EFFECT—GOOD CONDITIONING
Clarifying basin effluent	8	1.5
Rapid filter effluent	3	0.05
Slow sand effluent	0.1	0.02*

* Estimated.

During the past winter it was decided to see whether the initial high pressure aerator could be operated during the entire winter, a thing that had never been attempted before. A 10 foot fence was built around the aerator tray to prevent the wind from striking the spray and all ten lines of the header pipe were opened, thus cutting the height of the jets from 12 down to 3 feet. It will thus be seen that the aerator provided a maximum amount of agitation with the least amount of aeration. Two men were stationed at the aerator for each eight hour shift to break up the ice that built up around the nozzles and prevent the drift ice from clogging the outlet. Following the aeration the water enters the fifteen minute spiral flow tank and to break up the continuous ascending lines of uniform spiral currents, two air jets actuated by a Schutte and Koerting water ejectors were liberated at the bottom of the tank.

The results obained by this conditioning procedure were marked. The flocs were large at all times during the winter and the runs of the

slow-sand filters were normal at thirty days between scrapings. At no time during the past winter has the plant been unable to deliver its maximum output of 35 m.g.d., a thing never before experienced since the plant was started in 1899. The effect of conditioning on the floc turbidities throughout the plant is shown in table 7.

Plant D of table 6, treating the Hudson River at Waterford, N. Y. was noted for its passage of colloidal floc through the rapid filters during the winter months following the time it was placed in operation in 1915. This plant has been constantly worked with, and the process improved, until at the present time it turns out one of the best effluents of any of the plants with which the writer is connected. The benefits effected have been along the line of improved coagulation and have embraced pH correction by acid and improved conditioning by baffles and air jets. In the discussion of this paper, Mr. R. Gordon Yaxley, the Superintendent of the Waterford Plant, will go into these betterments more in detail.

TYPES OF PRESENT DAY CONDITIONING PROCESSES

Modern types of conditioning basins employ the following types of agitation to increase the size of the floc particles:

(1) Baffles. Include over and under and around the end types.
(2) Mechanical agitation. Includes high speed impellors either single or double, high speed followed by low speed paddles; variable speed proportioned area paddles.
(3) Spiral gravity flow agitation. Includes plain flow tanks and tanks having supplemental air jet agitation.
(4) Compressed air agitation.
(5) Agitation combined with aeration.

Baffle type agitators

This type is the oldest form of conditioning. The first large plant to use it was New Orleans in 1909. Whether over and under or around the end baffles are used depends upon the individual plant conditions. If the head does not fluctuate much the over and under type may be used. When basin levels fluctuate greatly the around the end type are preferable. Baffling may be made to attain any velocity or period of conditioning that any particular water may demand. By proper spacing of the baffles, the requisite velocities are attained and if the baffles are held in position by temporary bracing the velocities may be modified as changed plant conditions demand.

An example of flexibility in baffle design is offered by the new Helderberg Plant of the new Albany water supply. Here, the baffles are of one inch plank placed in the slots, overlapping like louvres, the entire section of one baffle being held at the top by one wedge. If the baffle is to be relocated, the wedge is knocked out and the entire assembly falls out of the slot which is loose fitting. Care must be exercised in the design of the spacing intervals that the suspended matter does not deposit out in dead pockets, in other words the velocities used must be sufficient for self cleaning. Velocities seldom fall below 0.3 or over 1.0 foot per second. Where there is head to spare it is probable that this mode of agitation is as economical as any, especially in northern climates where the basins must be housed, that is, the tanks being low may be used as substructure foundations for chemical housing, wash water tanks and the like. Most of the modern large plants employ this type of agitation although of course, there are exceptions.

Mechanical agitation

Mechanical agitation may be said to be a direct development of laboratory procedure. Its first application on a large scale was that developed by Langelier in connection with the Sacramento Filters in 1921. The principle involved is the production of velocity in a tank by means of a mechanically operated rotor. The power used may be water or electric. Water power is preferable, in that the speed of rotation of the rotor may be regulated efficiently. An interesting reciprocating water motor was developed at Sacramento for agitating each of the four tanks at varying velocities. Electric power requires reduction gearing or a slip-ring motor and pulley drive. Mechanical agitation admirably adapts itself to those installations that gave insufficient head available for the operation of baffled basins. Generally not more than two horse power is required to agitate a single unit. The tanks may be steel or concrete and can be readily enclosed within the chemical building close to the dosing apparatus when necessary. Mechanical agitation is especially applicable to old plants undergoing renovation. Where there is not room within the building for the tanks the agitators may often be arranged in compartments in the first pass of the clarifying basins. The work being done at Richmond, Va., is an interesting illustration of what may be accomplished in the rejuvenation of plants that have been in operation for some time.

When agitation tanks are used in series the speed of the rotors in

the different tanks may be varied, thus the speed of the initial agitator may be the highest to better agglomerate the floc, and the speed of the successive agitators reduced to bring about the requisite conditioning. A distinct advantage of mechanical agitation is that the velocity may be easily regulated to conform to varying plant outputs.

Mechanical agitators work on the continuous principle, the water generally entering the bottom at the side or middle, and leaving at the top either at the side or at the centre through a special collecting trough. The latter way is the better, as it tends to circumvent short-circuiting of the flow. The velocity set up within the tank is assumed to be a function of the peripheral velocity of the paddle, but it is at this point that the efficiency of the process is at question. In other words, the velocity of the water varies at different parts of the tank. This corresponds to the lag in velocity suffered in dead pockets of a baffled tank. There appears to be two zones of velocity in connection with the paddle type agitator: an inner and an outer one. The inner zone occurs from the shaft to about one third out; and the outer zone from there to the walls of the tank. Conditions may obtain where the velocity is too low at the inner zone and no floc will be formed, while a satisfactory floc will be formed in the more rapidly moving outer zone. At Sacramento this effect of varying velocity within the tanks was taken care of by proportioning the area of the paddle surface so that it was 25 percent of the vertical cross-sectional area. In this way uniform velocities were obtained. The individual paddle unit was made up of several members of varying size, the combined area of which was 25 percent of the sectional area. The paddles were hinged so as to lessen the resistance when the motors were started.

Many forms of rotors have been used in connection with mechanical agitation. High speed single and double discs have been employed. Some types have used a form of inter-locking paddle but this has generally induced too high a velocity. The variable speed, slow moving paddle type has been generally found to be the most satisfactory.

Spiral flow agitation

In spiral flow agitators the velocity head of a stream water is utilized in imparting the spiral flow, generally upward in a cylindrical concrete or steel tank. In the latter case the flow is generally under pressure. This type of agitator requires very low operating heads,

generally under one foot, and is especially well adapted to waters coming from an impounding reservoir to a treatment plant under pressure. The water enters the tank tangentially at the bottom and leaves in a like fashion at the top. The disadvantages of this process are: the velocity varies from zero at the centre to a maximum at the periphery; and also with the quantity of water being treated. These disadvantages are sometimes corrected by the insertion of fixed proportional wickets across the tank which add materially to the loss of head; or sometimes air is liberated from jets located at the bottom under moderate pressure. The small bubbles of air rising across the entire cross-section tend to break up the uniform spiral currents.

This type of agitation is used a great deal with clear waters not requiring too long a period of conditioning. Generally, fifteen minutes at an average velocity of from 0.5 to 0.75 feet per second is adequate. The process is especially adaptable as a prolonged conditioning for alum treated waters that have already received a preliminary conditioning through an aerator.

COMPRESSED AIR AGITATION

Floc conditioning by air is comparatively new and has a distinct field of possibilities. It is especially valuable as a means of conditioning in old plants never before having possessed this process. The air may be introduced in the first pass of a clarifying basin or along a flume at several points. New installations are generally designed in this fashion. Air developed by means of a low pressure blower, discharging through a manifold pipe which has several branches controlled by valves, is a most elastic means of obtaining economical agitation. The branches are spaced about six foot on centres and end with a tee pipe extending across the bottom of the flume. The tee branches are pierced with $\frac{1}{8}$-inch holes on 2 inch centres. Velocities of under one foot per second are induced by the rising air bubbles, and in addition to this a rolling motion of the whole volume of water is established by the liberation of the air at the bottom.

A compressed air installation on a large scale is illustrated by the plant at St. Paul, Minnesota. Here, the air in large volume is introduced into the entrance valve to a flume chamber just after the alum has been introduced. The reaction period is sufficient to condition the floc, and the diffused air serves to aerate the water as well.

To sum up, the advantages of compressed air agitation are:
(1) Low first cost and east of installation.
(2) Adaptability to old installations.
(3) Aeration combined with floc conditioning.

Combined aeration and agitation

Many large purification plants treating clear polluted waters have combined agitation with preliminary aeration. That is, the alum is added immediately to the incoming water and the aerator performs two functions: the reduction of the free carbon dioxide and the initial conditioning of the floc. Inasmuch as many of these waters require a low reaction pH for floc formation, the question has been raised as to the desirability of raising the pH by the reduction of carbon dioxide, so soon after the addition of the alum. It was argued that the low optimum reaction pH should be allowed to persist throughout the entire subsequent conditioning period for the proper formation of a settleable floc. The answer to this is, that in as much as the reaction between alum and alkalinity is practically instantaneous, the mixing period provided by the pumps and conduit piping to the aerator generally provides sufficient period for the actual floc formation even though it may be in a colloidal state. The conditioning provided by the aeration and subsequent forms of agitation may well proceed at the higher pH. In practice it has been found wise to provide secondary agitation at a reduced velocity to supplement the aerator conditioning. The spiral flow tank agitators lend themselves admirably to such a scheme. Where primary aeration is necessary this type of conditioning is practical and economical.

HYDRAULIC JUMP AGITATION

The hydraulic jump as developed by Ellms (10) for mixing and conditioning is an ingenious application of a well known hydraulic phenomenon to water coagulation practice. Inasmuch as the reaction period provided by the jump is little over thirty seconds it would appear that this device was essentially one for providing mixing of the chemical solutions in the actual sense of the word, and that subsequent low velocity conditioning would be necessary.

The jump is effected by the installation of a constricted flume, built on a batter, and receiving the discharge from the raw water pumps. The jump takes place at the end of the slope and the water

TABLE 8

Typical conditioning installations

PLANT	CAPACITY (M.G.D.)	DATE BUILT	TYPE OF AGITATION CONDITIONING	CONDITIONING VELOCITY (FEET PER SECOND)	RETENTION PERIOD (MINUTES)
Akron, Ohio	50	1915	Around end baffles	1.75	1
Albany, New York, Helderberg Plant	40	1932	Over under, round end	0.9	20
Atlanta, Georgia	42	1923	Around the end	2.0	25
Baltimore, Maryland, New Plant	112	1928	Around the end	1.3	30
Birmingham, Alabama	43	1924	Mechanical	0.7	10
Buffalo, New York	160	1926	None except in pumps		
Cedar Rapids, Iowa		1930	Mechanical and air	1.5	30
Cleveland, Ohio, Division Plant	140	1918	Over and under	0.4	39
Cleveland, Ohio, Baldwin Plant	150	1924	Hydraulic jump, over under	10.0	1
Denver, Colorado	60	1924	Around end	0.3	20
Detroit, Michigan, Water Works Park	320	1923	Around end	1.3	3
Detroit, Michigan, Springwells Plant	272	1931	Mechanical	0.4	17
Erie, Pennsylvania	32	1925	Vertical baffles	0.1	19
Evansville, Indiana	18	1931	Over and under		7
Flint, Michigan	28	1924	Vertical baffles	0.6	35
Fort Worth, Texas	20	1923	Around end	0.5	40
Grand Rapids, Michigan	40	1924	Around end	1.0	45
Kansas City, Kansas	25	1927	Around end	1.1	10
Kansas City, Missouri	100	1928	Spiral flow		30
Knoxville, Tennessee		1927	Mechanical	1.5	20
Miami, Florida	20		Mechanical	1.0	20
Minneapolis, Minnesota	40	1928	Over and under	0.7	3
New Orleans, Louisiana	112	1928	Around end	0.8	60
Norfolk, Virginia	12	1923	Over and under	0.7	10
Oakland, California	12	1927	Mechanical	1.7	20
Oklahoma City, Oklahoma	16	1923	Over and under	0.6	30
Providence, Rhode Island	48	1926	Spiral flow	4.6	5
Richmond, Virginia	30	1924	Over and under	0.8	10
Spartanburg, South Carolina	6	1925	Over and under	0.6	30
Springfield, Illinois	12		Mechanical	0.8	
St. Louis, Missouri	80		Spiral flow	0.7	16
Tampa, Florida	14	1926	Mechanical	0.8	100

TABLE 8—*Concluded*

PLANT	CAPACITY (M.G.D.)	DATE BUILT	TYPE OF AGITATION CONDITIONING	CONDITIONING VELOCITY (FEET PER SECOND)	RETENTION PERIOD (MINUTES)
Tulsa, Oklahoma	24	1930	Spiral flow and water pressure		
Washington, D. C., Great Falls	80	1927	Around end	1.5	27
Wheeling, West Virginia	14		Over and under	0.6	60
Wilmington, Delaware	12	1926	Around end	0.4	7
Windsor, Ontario	21	1926	Over and under	0.8	4

Several of the above data taken from a chart compiled by Eugene A. Hardin, Detroit Water Works.

boils over on itself as it seeks its initial elevation, it is thus apparent that a chemical solution added just before the jump takes place receives the maximum amount of diffusion possible. The jump actually requires one foot of head to operate it but it has been found in practice that due to fluctuations in clarifying basin elevations, about two feet available head is required.

In table 8 are listed thirty-seven typical large mechanical plants. Of these, 2.7 percent are conditioned by air, 2.7 percent by hydraulic jump, 10.8 percent by spiral flow, 21.6 percent are mechanically agitated, and 62.3 percent depend upon the velocity induced by baffling. It is thus seen that baffling is the more popular form of agitation, although there is a distinct trend towards mechanical conditioning in modern plants.

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DISCUSSION

R. G. YAXLEY:[2] At the start of this discussion I wish to most heartily endorse the statements made by Mr. Willcomb in his opening paragraphs. I specifically refer to the indiscriminate use of the word "mixing." Webster's definition of mix is "to unite or blend into one mass or compound," yet in technical papers covering the preparation of water for a rapid sand filter we frequently find this word being used when any informed reader knows that the writer had something entirely different in mind. Mixing is the art or practice of distributing the coagulant throughout the mass of water being treated. Conditioning is so apt and proper a term for use in connection with the present subject, that there is no excuse for confusing the two words.

Mr. Willcomb's comment on winter troubles at the Waterford plant was but too true. Heavy alum feeds, cold water and short detention periods in an old style plant surely spell trouble. Our original layout consisted of two rectangular basins operating in parallel and each provided with a single concrete baffle wall with a submerged port at the farther end, in effect a single round the end baffle, dividing each basin into two equal parts. Additional wooden round-the-end baffles were installed which divided each basin into eight equal parts. This cut out short circuiting and rendered the entire basin area effective. Next the plant was put under strict pH control, using sulphuric acid as a correcting medium. The addition of 0.5 grain of acid reduced the alum feed from 3.5, an average for a five year period, to 1.84 grains also a five year average. With this marked reduction in the amount of alumina applied, things began to look brighter in the clear well. The next step was the use of air jets in the first and second pass of each basin. Each pass is supplied by a single No. 182-XL 96 Pemberthy water operated ejector, the discharge of which is a plain open end pipe fitting located in the approximate center of a basin subdivision and submerged about eleven feet. No attempt was made to diffuse the air throughout the section as maximum air lift action was desired to obtain a rolling motion in the body of water. Each basin pass has a detention period of between 15 and 20 minutes and the air is used in the first pass at all times. The second pass jets are used only during cold weather, the shorter period being sufficient for warmer water. The total installation cost of the air equipment was about $60.00 and the operating cost is less

[2] Superintendent to Water Commissioners, Waterford, N. Y.

than five gallons of water under 80 pounds pressure per minute for each jet.

Results obtained are almost unbelievable unless actually observed. By closing off the air from one basin, while the other has the regular supply, a perfect check can be obtained as all other conditions remain identical. Tests of this character conducted in midwinter show a reduction of over 30 per cent in effluent turbidity in the basin using the air. Filter runs have been more than doubled and Mr. Willcomb's figures on Al_2O_3 in the filter effluent speak for the general efficiency.

The author also mentions a home made mixing machine for laboratory use. This was constructed out of material found around the plant, except for the reducing gear and a few screw eyes which cost less than $10.00 and the "pickle jars." These were quietly removed from the pantry shelf when the Mrs. was out. A picture of the machine will be found, accompanying an article by Mr. Cox, in the March 9, 1932, issue of Water Works Engineering. I find this machine not alone useful for determining the proper alum dose and optimum pH, but it is indispensable for finding the proper conditioning period. It is also handy when some salesman appears with a new fangled coagulant, just ask for a small sample and try it without going to all the trouble of rigging up for a plant sized test.

One of the outstanding features of this paper is the author's method of measuring floc in terms of turbidity and the ingenious method devised to obtain accurate readings of the low values encountered in such work. All experienced workers know the difficulty of obtaining reasonably accurate readings when dealing with a turbidity of a fraction of a p.p.m. Heretofore the personal element alone has been so important that different persons in the same laboratory sometimes would not agree within 100 percent, so it is no wonder that little value was given to comparisons between plants, but here is a method that at least approaches a standard. Using the same type of glass ware and giving a reasonable attention to the lighting effects, such as always working before windows with northern exposure, operators in different plants should be able to prepare data that would be of some use. The real value of the method, however, seems to me to be in its simplicity which allows its adoption as a routine plant test. This is not practical with some of the other methods owing to the time and fussing required to obtain even an approximate determination.

In conclusion, I think it is conservative to say that Mr. Willcomb has rendered a real service to the craft in preparing this paper, as very little information on this important subject has been available, in the past, to the general plant operator. Only those who have prepared similar papers can appreciate the amount of work he has performed for the benefit of his co-worker in the water works field.

GEORGE B. PRINDLE:[3] We are indebted to Mr. Willcomb for pointing out the incongruity of the term "mixing" as applied to the process of agglomeration. As he states, the mixing of the coagulant with the raw water, and the consequent formation of colloidal floc, are almost instantaneous. The subsequent agitation of the water does not create additional floc, but brings together into larger masses the minute particles already formed. As this process is one of agglomeration, it would seem there could be no better term for describing it. The term "conditioning," suggested by Mr. Willcomb, is broad enough to apply to any process altering the character of the water.

To change the name of the settling basin would not seem so logical. Whether suspended matter in water results from the action of wind and wave, or from the ingenuity of the chemist, the process by which it is settled out is certainly that of sedimentation. "Clarification" would seem a term broad enough to include the function of the filter bed. The relatively few basins in modern practice for the sedimentation of water before chemical treatment reduces the importance of the distinction.

In this connection, however officially the chemists and engineers may adopt glossaries descriptive of the works they design, the men who build them, and the operators who follow will probably continue to describe the various units in the easiest way. When the last filter plant is constructed, the reservoir for the storage of filtered water may—on the plans—have a very high sounding name, but the operator will call it his "clear well" just the same.

Mr. Willcomb is to be congratulated on his scholarly treatment of the theory of the formation of floc, and for the description of methods for its measurement. His turbidity standards made of clusters of microscopic-cover-glass should prove very useful. Between the maximum range of the Baylis and the minimum range of the Jackson turbidimeters there has been a "no man's land," which the silica standards covered in an unsatisfactory way.

[3] Superintendent Water Works, Highland Park, Ill.

In connection with the discussion of mechanical agitators, the speaker has for the past two years operated four of them in the 7 m.g. filter plant at Highland Park, Illinois. These works are located in a residential suburb of Chicago, and handle Lake Michigan water, generally considered difficult of treatment. The coagulation secured has been very satisfactory, resulting in good filter runs and extremely low turbidities of the finished water. These basins which have been described and illustrated by Baylis (1) have a nominal retention period of 50 minutes. Observation of their operation gives emphasis to the important function which eddy currents play in the process. If they had been circular in shape, the motion of the paddles would have set up a rotation of the entire mass of water resulting in very little stirring action. As the basins are practically square in horizontal cross section, the rotation of the mass of water is interrupted by the corners, and very evident eddy currents result. These eddy currents produce a weaving action of the water, gently bringing the particles of floc into contact with each other, thus creating an ideal condition for agglomeration.

In basins with mechanical agitators of this type it would seem that the speed of rotation of the paddles is a function of the mass of water. As this does not change appreciably with changing rates of pumpage, we have not found it desirable to alter the speed of the paddles with pumpage variations.

SEDIMENTATION BASIN RESEARCH AND DESIGN

By Arthur B. Morrill[1]

Knowledge of what goes on in sedimentation basins and of the basic principles by which they should be designed is most inadequate. Strangely enough, in view of the remarkable advances made in so many other fields of water purification, there has been very little advance in this direction for a quarter of a century. There have been important advances associated in one way or another with sedimentation basins, such as the introduction of mechanical sludge removal, but as to the theory of sedimentation the water works engineer has hardly moved from the spot where he was left after the publication of Allen Hazen's important paper[2] in 1904.

Sedimentation is taken here to mean either with or without the use of coagulants. Plain sedimentation is still in common use, but sedimentation after coagulation is a process quite as important, and probably more so, in modern water purification. All recent American works treating the subject present without change Hazen's theory of sedimentation. This is done in a way which implies that the theory is valid for all kinds of sedimentation basins, including coagulation basins. Reference to Hazen's original paper will show that the theory does not apply to coagulation basins and that the author fully appreciated that fact. This matter is considered further below.

Hazen's main conclusions were based entirely on mathematical analysis. There are no published reports of attempts to confirm the theories experimentally. While a great deal of experimenting has been done on other parts of the water purification process, very little has been done on sedimentation. This is, of course, due to the inherent difficulties of sedimentation experiments, but the fact remains that the modern designer has only imperfect ideas of what means must be provided to produce a certain effect in a sedimentation basin.

[1] Assistant Engineer of Filtration, Department of Water Supply, Detroit, Mich.

[2] On Sedimentation, Allen Hazen, Trans. Am. Soc. C. E., 1904, p. 45.

HYDRAULIC MODELS

Experimental investigation of sedimentation basins is for certain purposes fairly simple and for other purposes very troublesome and expensive. What may be called the hydraulic action of a proposed basin may be investigated by the use of a scale model. In certain cases, at least, this kind of investigation will not be unduly expensive and the results obtainable may be used with confidence. Largely through the efforts of John R. Freeman, American engineers have learned during the last few years of the great value of hydraulic models. Such models should be much more generally used in the design of important sedimentation basins.

Until recently sedimentation experiments have generally been made in small basins in which the detention time was the same as that proposed for the full-sized basin. These have usually been studies of sewage sedimentation rather than of water sedimentation. For hydraulic studies model basins so operated are useless. If a large basin and its model are geometrically similar and n is the ratio of their linear dimensions, then the volume of the large basin will be n^3 times that of the model. Its length will be n times the length of the model. If the nominal detention time is the same for prototype and model, then the velocity in the large basin will be n times that in the model. If on the other hand the velocity is made the same in model and prototype, then the detention time is n times greater in the large basin.

For "dynamical similarity" it has been shown[3] that the velocity in the prototype must be $\sqrt{n}$ times the velocity in the model. The detention period in the prototype is then also $\sqrt{n}$ times that in the model. If these conditions are met the hydraulic effects produced in the large basin will be the same as those produced in the model.

For example, suppose it is desired to study the operation of a proposed basin 200 feet long, 50 feet wide and 15 feet deep. If n, the scale ratio, is taken equal to 9, then the model will be 22.2 feet long, 5.55 feet wide and 1.66 feet deep. If a 2½-hour detention time is proposed for the large basin, that to be used for the model is $2.50 \div \sqrt{9} = 0.833$ hours. The nominal velocities will be 1.33 feet per minute in the large basin and $\frac{1.33}{\sqrt{9}}$ or 0.444 feet per minute in the model.

[3] Hydraulic Laboratory Practice," Freeman, 1929, also "Hydraulics and Its Applications," Gibson, 1925, p. 71.

The practical effect of dynamical similarity in sedimentation basin research is that the distribution of flow and the eddies produced in the model are similar to what will be produced on full scale. If the above rules are disregarded and the detention time is made the same in model and prototype, the distribution of flow may be entirely different and the results very misleading. It is possible to construct and operate a scale model in such a way as to give valuable information as to the distribution of flow which will result from a proposed design. Eddies can be observed and means found for largely eliminating them. In this way the hydraulic action of the large basin can be predicted.

Hydraulic model research has certain inherent limitations. Particularly with large scale ratios, that is, unusually small models, effects of viscosity are troublesome and sometimes fatal. The effects of surface tension are disproportionately large in a model. Above certain limits of velocity and size this fact is not serious enough to have much effect on the results, but under other conditions anomalous results may be caused. In a model test[4] of the intake lagoon of the new water supply of Detroit the scale ratio was 33 to 1. The model was about 27 feet long, 15 feet wide and 0.80 feet deep. At times dust would be blown onto the surface of the water. Under certain conditions the surface of the water in the model would be quite motionless, even though the water below was moving with an average velocity of 1.2 feet per minute. Surface tension and the dust on the water caused a phenomenon which could not possibly occur on full scale. Tube floats in the water would not move, although their lower parts were in moving water. Their upper ends were stuck fast in the surface film, just as if a sheet of ice covered the lagoon.

In model investigations for the Detroit sedimentation basins the scale ratio was 20 to 1. The mean velocities were between 0.4 and 2.0 feet per minute. The model was indoors and so protected from dust accumulations. A photograph of the model basin during a test run is shown in figure 1. No troublesome effects of viscosity or surface tension were observed. In comparing the model with an existing basin the hydraulic effects appeared to be faithfully reproduced.

IDEAL CONDITIONS FOR SEDIMENTATION

In any cross-section of an ideal settling basin the velocity would be uniform. The velocity near the sides would be the same as at the

[4] Huge Intake Lagoon Built to Avoid Ice Troubles, Torris Eide, Eng. News-Record, Sept. 24, 1931.

center. Near the bottom the velocity would be no less than at the sides. Most designers will agree to the statement regarding side velocity, but some may disagree as to velocity near the bottom. It has been urged that sedimentation will be better if there is a zone of quiet water near the bottom to receive sludge. The fact seems to be disregarded that, if some parts of the water move at less than the nominal velocity other parts, to compensate, must move at greater velocities. If a basin has a nominal detention time of three hours and

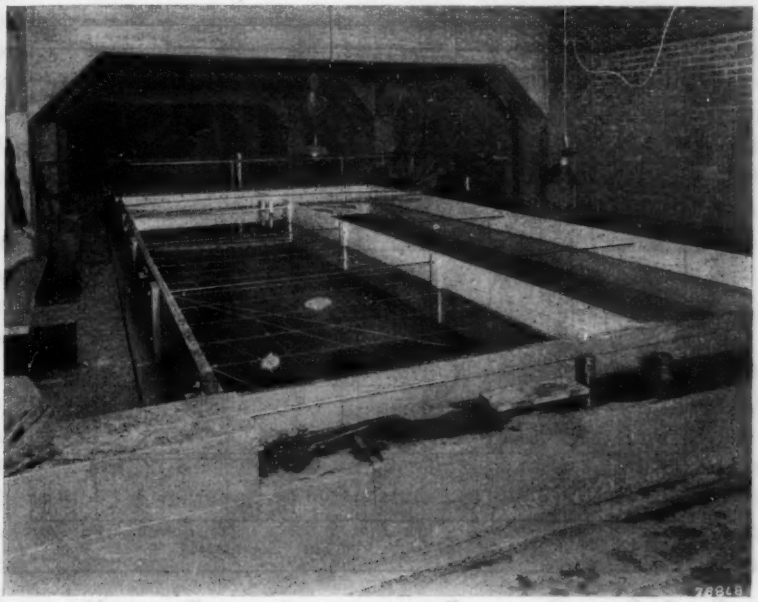

FIG. 1. SCALE MODEL USED IN DESIGNING DETROIT SEDIMENTATION BASINS

the lower third is practically stagnant, then the actual average detention time must be two hours. There is good reason for believing that the quiet zone near the bottom is quite unnecessary and represents a loss in basin capacity.

It has been stated by Imhoff[5] that sedimentation in smoothly flowing water, provided a certain velocity is not exceeded, goes on just

[5] Zur Berechnung der Absetzbecken, K. Imhoff, Gesundheits-Ingenieur, June 20, 1925, also Design of Settling Basins for Sewage Treatment Plants, K. Imhoff, Water Works, June, 1928.

as in still water. What this limiting velocity is has not been determined, but Imhoff states that it is surely at least 10 feet per minute. It will always be possible to make designs such that the average velocity will be well within this limit, so the effect of velocity on sedimentation can be disregarded.

The important thing, in sedimentation in a basin of given size, is to insure that every particle of water passing through the basin is treated the same length of time, insofar as this is possible. Designers sometimes ease their consciences with the thought that if some of the

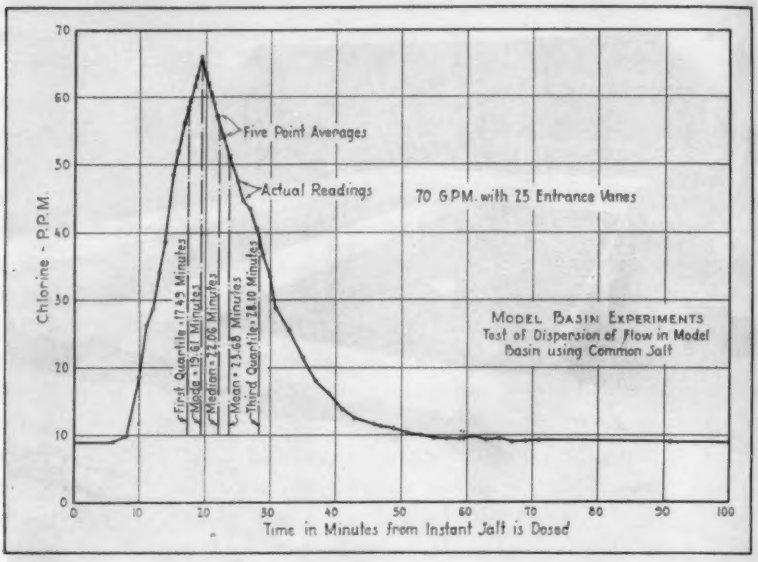
Chlorine - P.P.M.
Time in Minutes from Instant Salt is Dosed
Five Point Averages
Actual Readings
70 G.P.M. with 25 Entrance Vanes
First Quartile = 17.45 Minutes
Mode = 19.61 Minutes
Median = 22.06 Minutes
Mean = 23.68 Minutes
Third Quartile = 28.10 Minutes
Model Basin Experiments
Test of Dispersion of Flow in Model Basin using Common Salt

FIG. 2. CURVE SHOWING DISPERSION OF SALT IN TEST OF MODEL SEDIMENTATION BASIN

water is settled for much less than the average time, other parts must be treated for more than the average time, and the final result may not be seriously affected. Since the first hour of settling is always very much more useful in improving the quality of a water than later periods of the same length, any lack in uniformity of treatment must lower the efficiency of a sedimentation basin.

DISPERSION TESTS

In model basins, and sometimes on full scale, distribution of flow can be shown in a practical way by introducing a dose of chemical

salt or coloring matter into the influent. In the chemical method common salt is generally used. Frequent samples of the effluent are taken and titrated for chloride. Plotting the concentration of chloride against the time of sampling, a graph can be drawn which gives a good idea of the action of the sedimentation basin. Such a curve is shown in figure 2.

The salt method has difficulties. The brine introduced is decidedly heavier than water. If the velocity is low, some of the brine may settle to the bottom before it has time to become mixed with the water with which it entered. This difficulty can be overcome in part by using ammonium chloride instead of common salt. For the same concentration of chlorine the ammonium chloride gives a solution of lower specific gravity. Another difficulty is that the titration of dozens or hundreds of samples is laborious, and none too precise unless a large amount of salt is used. The chloride content sometimes fails to return to the starting point. This may occasionally be due to a fluctuation in the amount of chloride naturally present in the water. It is more apt to be due to the settling of part of the brine and its slow diffusion back into the moving part of the tank for many hours after it should have disappeared.

Chemical analysis of a large number of samples may be avoided by the use of a conductivity indicator. A conductivity cell is inserted in the stream of water flowing out of the basin and the electrical resistance can be read as frequently as is desired without even the trouble of sampling. The resistance of the cell is inversely proportional to the conductivity of the water and conductivity increases in direct proportion to the concentration of salt present in the effluent at the time.

Interesting qualitative results may be obtained quickly in model basins by the use of dyes such as uranine. At Detroit the light conditions were such that uranine was not satisfactory, but very good results were obtained with potassium permanganate. With coloring matter the reason for bad distribution of flow in a model basin can often be seen at a glance. After inlet devices or vanes have been added or changed until the distribution appears to be excellent, a "salt run" may then be made to give a quantitative measure of the effect.

The salt run is an old device for testing the detention time in a settling basin. It appears, however, that very little use has been made of the great quantitative possibilities of such tests. By proper

analysis there can be obtained from such tests a numerical measure of dispersion in a settling basin, and dispersion may be an important factor in determining the efficiency of the basin in service.

DISPERSION DEFINED

In an ideal settling basin all the water which enters the inlet during one second will leave the outlet during another second, some hours later. The elapsed time between these two instants will be equal to the detention time of the basin. In actual basins the water that enters the inlet during any second is spread out over a broad

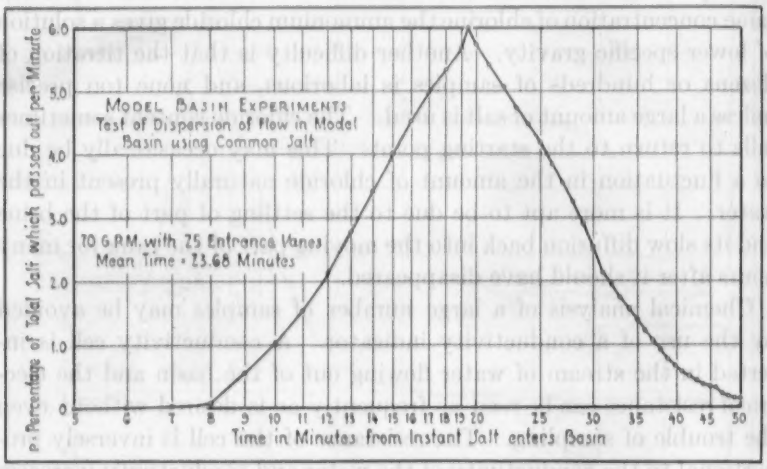
MODEL BASIN EXPERIMENTS
Test of Dispersion of Flow in Model Basin using Common Salt
70 G.P.M. with 25 Entrance Vanes
Mean Time = 23.68 Minutes
p: Percentage of Total Salt which passed out per Minute
6.0
5.0
4.0
3.0
2.0
1.0
0
5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 25 30 35 40 45 50
Time in Minutes from Instant Salt enters Basin

FIG. 3. DISPERSION TEST CURVE WITH TIME PLOTTED ON LOGARITHMIC SCALE

band of time before it reaches the outlet. Figure 2 shows a salt test of such a basin. The nominal time is 23.7 minutes. The first salt appears at the outlet, however, in 8 minutes, the maximum concentration is reached in 19.6 minutes, and traces of the salt have practically disappeared in 55 minutes.

The name "dispersion" is suggested for this spreading of the water, in terms of time, ahead of or behind the time position it would occupy in an ideal basin. Dispersion is always objectionable in a settling basin, it should be reduced to the lowest practical limits and the sedimentation efficiency of a basin will always be improved as the dispersion is reduced.

A numerical expression of dispersion is needed. Mathematically a salt-run test curve is a skew frequency curve. If the center of gravity of the area under the curve is located, the abscissa of this point is equal to the nominal detention time of the basin. The peak of the curve has, in statistics, an interesting name. It is called the "mode." The mode time is always less than the mean time. Possibly the difference between these two times could be used as a measure of the dispersion. There appears, however, to be a better means of measuring this characteristic.

Another interesting mathematical characteristic of salt test curves is that, in some cases at least, they become symmetrical instead of

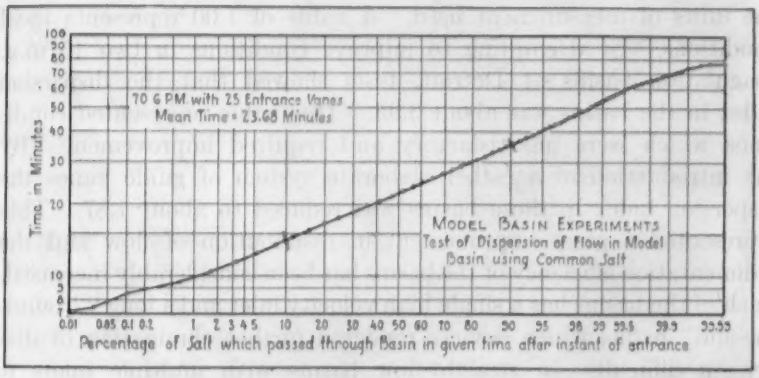
70 6 PM with 25 Entrance Vanes
Mean Time = 23.68 Minutes
Time in Minutes
MODEL BASIN EXPERIMENTS
Test of Dispersion of Flow in Model
Basin using Common Salt
Percentage of Salt which passed through Basin in given time after instant of entrance

FIG. 4. CUMULATIVE DISPERSION CURVE PLOTTED ON LOGARITHMIC PROBABILITY PAPER

skewed when the horizontal scale is made logarithmic. If the curve of figure 2 is replotted on semi-log paper, with the logarithmic scale for time, it reduces to a nearly perfect isosceles triangle, as shown in figure 3.

In figure 4 the data of figure 2 have been replotted in what may be called a cumulative curve. The ordinates represent time, on a logarithmic scale, since the salt was added to the incoming water. The abscissas show what percentage of the total quantity of salt had passed out of the basin at the time in question. These percentages are plotted on the probability scale devised by Allen Hazen. It will be noted that while the line is somewhat curved at the ends, it is nearly straight throughout the interval from 10 to 90 percent.

This is due to the use of the probability scale and makes interpolation easy and accurate.

It will be seen that the slope of the curve in figure 4 is a measure of the dispersion in the basin. In an ideal basin without dispersion, the curve would be a horizontal straight line. The steeper the slope of the line, the worse the dispersion. If the 90 percent time, taken from the curve, is divided by the 10 percent time, the result becomes a numerical measure of the dispersion. It might be called the "dispersion index." From figure 4 the 90 percent time is 34.8 minutes, the 10 percent time is 14.1 minutes and so the dispersion index is 2.47.

This dispersion index is a pure number or ratio, independent of the units of measurement used. A value of 1.00 represents ideal conditions. In attempting to improve conditions in two 15 m.g. coagulation basins at Detroit, tests showed that the dispersion index in the basins was about 3.65. This value represented conditions which were unsatisfactory and required improvement. By the introduction of a rather elaborate system of guide vanes the dispersion index in these basins was reduced to about 1.87. This represented a great improvement in distribution of flow and the sedimentation efficiency of the basins has been considerably increased. Each of the basins has a single high velocity inlet and a long " 'round-the-end" baffle, which devices rendered further elimination of dispersion difficult. In straight-flow basins with multiple inlets it should be possible to reduce the dispersion index to about 1.7 or less. In the model tests for the new Springwells plant at Detroit a dispersion index of about 1.67 was obtained without unduly elaborate arrangements.

Waterworks men may recognize that the proposed dispersion index is a characteristic very similar in its nature to the uniformity coefficient used to describe filter sands, except that the 60 percent and 10 percent points on the curve are used for filter sands, while the 90 percent and 10 percent points on the curve are suggested for the dispersion index. It might be called a "uniformity coefficient for sedimentation basins," in that its value indicates the uniformity of treatment which the water flowing through the basins is receiving.

In all the foregoing discussion the emphasis has been placed on dispersion, which after all is a sort of negative attribute. Lack of dispersion is the positive quality that should be sought, but there seems to be no simple and obvious name for this quality. Perhaps

"volumetric efficiency" would do, and this might be defined as the reciprocal of the dispersion index, the 10 percent time divided by the 90 percent time. The "volumetric efficiency" would then approach one or 100 percent as a maximum. With this definition the 15 m.g. basins mentioned would have had an original volumetric efficiency of 27.4 percent, which was raised to 53.5 percent by the introduction of guide vanes. A photograph of some of these guide vanes is shown in figure 5.

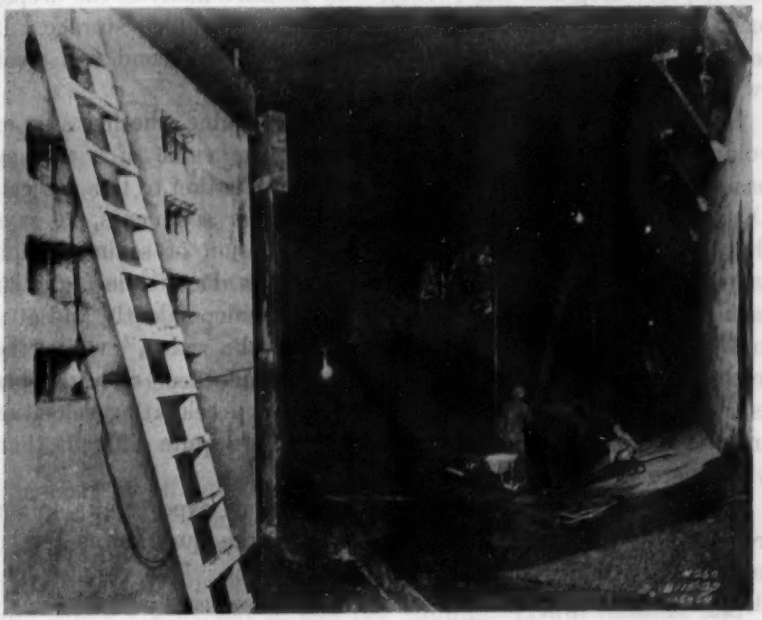

FIG. 5. GUIDE VANES NEAR ENTRANCE OF WEST COAGULATION BASIN OF FIRST DETROIT PLANT

SEDIMENTATION RESEARCH

So far the discussion of investigation and research has been confined to uniform distribution of velocity in every cross-section of flow. Another side to the question remains, perhaps more important than the first, but more difficult to investigate. This is the study of the basic principles of sedimentation. In what way is the sedimentation capacity of a basin a function of its length, width and depth? These matters do not appear to be so susceptible of experi-

mental investigation, certainly not in the models that are used for dispersion tests. Attempts have been made to test sedimentation in laboratory jars, but the translating of results is a doubtful process in the present state of knowledge.

Whereas Hazen's analysis appears to show that sedimentation depends only on surface area, not on depth, it is all but universal practice to rate sedimentation basins in terms of sedimentation time that is, in terms of volume. Consider two basins of the same area, one ten feet deep and one twenty. By Hazen's analysis the deeper basin is no better, except that it has greater sludge storage. If sedimentation time is the proper criterion, the second will treat twice as much water as the first.

It is of great practical importance to determine whether area or volume is the valid basis for estimating the value of a settling basin. Imhoff[6] has made an important contribution to this subject. Distinguishing between granular and floccular sediments, he says that, for the former, area is the true criterion of sedimentation capacity and, for the latter, volume. This is due to the fact that each particle of a granular sediment settles independently and at a uniform velocity depending on its "hydraulic value," while the settling particles of floccular sediments coagulate or clot together as the settling proceeds. Thus they settle at accelerating velocities and the analytical conclusions to be reached are quite different than when there is no coagulation.

A great deal of work was done at the Detroit experimental plant from 1927 to 1929 in an attempt to answer the question as to the relative importance of surface area and volume. The results were disappointing, largely because the raw water was usually clear enough to filter fairly well with no sedimentation at all. Other factors may have contributed; probably there were undetected differences in dispersion which influenced the results. It is to be hoped that similar experiments will be conducted elsewhere under more favorable circumstances.

Some heavy sediments obtained in water purification may be truly in the granular class. Light flocks obtained by coagulating fairly clear waters with alum appearing definitely in the floccular class. Other sediments may be intermediate, as Imhoff believes is generally the case with sewage sediments. There is need for investigation that will aid in classifying sediments in this respect.

[6] The Arithmetic of Sewage Treatment Works, p. 12, Wiley & Sons, 1929.

ELEMENTS OF SEDIMENTATION BASIN DESIGN

Disregarding all questions of structural design, the more important points to be determined in designing a sedimentation basin installation are the following:

1. Number of basins
2. Length
3. Width
4. Effective depth
5. Velocity of flow
6. Detention time
7. Sludge storage volume
8. Method of sludge removal
9. Inlet arrangements
10. Intermediate arrangements
11. Outlet arrangements
12. Basins open or covered

Number of basins

The number of basins is to be determined usually by questions of flexibility. To allow for one basin being out of service occasionally for cleaning or repair, the total number of basins should at least be three, except in small plants.

Dimensions of basins

The length, width and depth of each basin are to be so selected as to give the desired volume with the least cost. There are, however, certain limitations. The length and the nominal settling time at maximum capacity determine the maximum nominal velocity. This should surely not exceed 10 feet per minute. It will be safer and usually entirely practical, to keep this velocity down to about 3 feet per minute.

It will be easier to avoid dispersion, that is, to get good volumetric efficiency, if the length is at least three times the width. If the basins are made shallow, the cost will be increased, particularly if they are covered or if hopper bottoms are provided for removing sludge. Depths of 5 to 20 feet are found economical in plants of different sizes. The width of a basin is usually several times the depth, but the only reason for this appears to be that some such ratio is generally economical.

Detention time

The most important single point to be decided in the design of a sedimentation basin is the determination of the detention time. Practice has varied from an hour or less to several days. Many basins with long nominal sedimentation times have low volumetric efficiencies. Probably equally good results could be obtained with smaller basins, if more care were taken to avoid dispersion or short-circuiting. Softening, very high turbidities or algae trouble may require long sedimentation times. At the other extreme there are filter plants where there is some question whether sedimentation is justifiable at all. To meet a given demand, any slight increase in the size of sedimentation basin should make possible a slight decrease in the size of filter plant and vice versa. That combination should be selected which gives the lowest total cost, considering all charges. On account of the difficulty in determining the effect of a change in sedimentation time on necessary filter capacity, it is seldom possible to show definitely what detention time is most economical.

Sludge storage volume

Unless the sludge is to be removed continuously allowance must be made in the basin volume for the accumulation of sludge between cleanings. These are usually planned from three to six months apart. If more frequent cleanings are necessary to prevent the sludge encroaching unduly on the settling volume, then mechanical sludge removal is apt to prove more economical.

Designing for volumetric efficiency

Planning a settling basin to secure good volumetric efficiency requires the best ingenuity of the designer. Many basins are very unsatisfactory in this respect. Their deficiencies are serious, but not very easy to observe and measure, otherwise they would not be tolerated. A common way of attempting to secure good "displacement" has been to introduce baffles. In most places interior baffles do more harm than good. Perforated inlet or outlet baffles fall in a different category and may be used to great advantage. Particularly in very large plants baffling is fundamentally wrong. Baffles are less objectionable in very small plants, where velocities are apt to be low, but even here it seems that it should always be possible to secure better results by other means.

The essential characteristic of an interior baffle is that it obstructs

the flow of the water. It is certain to produce mixing and eddies. These are good for mixing chambers, but bad for sedimentation basins. If it is necessary to produce a change in direction of flow at any point, this should be done by guide vanes, not by baffles. The water should be guided, not obstructed.

The forces which produce motion of the water in a sedimentation basin are extremely small. Compare, for example the conditions of flow in a small pipe and in a sedimentation basin, each 100 feet long. The loss of head in the pipe under ordinary operating conditions will be a matter of inches, possibly even several feet, while the loss in the basin will be so small as to be quite undetectable by any surveying process. If the mean velocity is one foot per minute, the velocity head will be about one twenty thousandth part of an inch. The total loss of head from one end of the basin to the other, disregarding entrance losses will probably be considerably less than this.

On this account the production of anything like uniform velocity throughout any cross-section of a basin is a very delicate matter. In an open basin the effect of wind may entirely upset the distribution of velocity, even producing a reverse current at the surface or along one side of the basin. If the incoming water is decidedly warmer or cooler than the water in the basin, thermal stratification may result with serious disturbance of proper flow conditions. This factor does not appear to cause trouble in covered basins with a nominal velocity of 8 feet per minute. With very low velocities and in basins exposed to the sun such trouble is more apt to develop.

Inlet arrangements

The velocity of the water in the pipes or channels supplying the sedimentation basin may be from 20 to 100 times greater than in the basin itself, so that the kinetic energy of the water in the supply line may be 3000 times as great as that of the same water when it gets into the basin. This means that the greatest care must be used in reducing velocity and in getting the water started through the basin uniformly all over the cross section.

Of the various plans adopted, the perforated baffle appears the most promising. This arrangement was used some years ago at Toledo and doubtless at other places. A similar arrangement was used for the new Detroit plant, but the quantities involved were such that it was found better to use vertical slots than orifices. In this plan the incoming water is brought into a sort of stilling chamber

extending the full width of one end of the basin. This chamber must be large enough so that the maximum velocity in it is small, 30 feet per minute or less. The wall separating the stilling chamber is perforated with openings scattered over the full width and height of the basin cross-section. These openings must not be too small or the velocity of the water entering the basin will be too great and cause disturbance. They must not be too large, or the loss of head produced will be insufficient to smooth out inequalities due to slight differences in level in different parts of the stilling chamber.

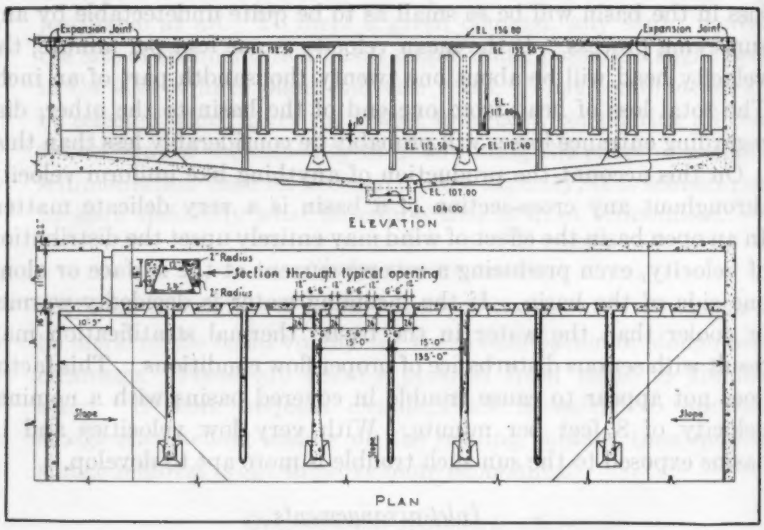

FIG. 6. DRAWING OF INLET END OF COAGULATION BASIN SHOWING INLET SLOTS AND VANES

To keep the size of the stilling chamber within practical limits it seems inevitable in most cases that the main direction of flow in it should be at right angles to the direction of flow in the sedimentation basin. The water tends to pass through the inlet baffle at an angle, producing a large eddy in the basin. After laborious attempts to prevent such an eddy by varying the size or spacing of the inlet slots, it was found in the Detroit experimental work much more effective to provide short training walls at the inlet end of the basin, extending for about 30 feet in a basin 330 feet long. Model tests showed that longer training walls were not enough better to compensate for their

greater cost. The arrangement of inlet slots and training walls adopted is shown in figure 6.

Outlet arrangements

As to outlet devices, the perforated baffle seems to be an excellent arrangement, but it is not as necessary here as at the inlet end. A weir, which is rather unsatisfactory as an inlet device, works well as an outlet device. Free-falling weirs will be satisfactory for small plants, but for large plants a submerged weir is much better as the outlet velocity is less. The weir should extend the full width of the outlet end of the basin. A submerged weir should be so designed that the loss of head will be small. A loss of one inch will be sufficient to give uniform draft at all points of the width and will give a lower outlet velocity than a greater loss of head. A skimming weir across the outlet end of the basin will often be advantageous to prevent scum passing on to the filters.

Covering sedimentation basins

Roofs over sedimentation basins have many advantages, but their cost is such that they are seldom used except where required to prevent freezing. As above stated, they prevent wind disturbance and thermal stratification. In small plants the full width of each basin unit may usually be spanned without the use of columns, but in very large plants this is sometimes impractical. For the new Detroit plant it was found necessary to use interior columns and a good deal of attention was given to the effect of these in disturbing the flow, developing eddies and increasing dispersion. On this account the spans of the concrete roof were made as long as possible. Model experiments were made on round columns and on stream line or ship-shaped columns. Color tests showed plainly that any columns in a basin are a decided disadvantage and that a roughly ship-shaped column is distinctly better than a round one. The hydraulic advantage of a special column shape was not felt sufficient, however, to overcome the considerable disadvantage in cost and structural complication, so round columns were used.

SUMMARY

There is great need for improvement in the design of sedimentation basins. Hydraulic models, properly designed with regard to dynamical similarity, will yield valuable information. Much more

attention should be paid to securing uniform velocity in any cross-section. Otherwise "dispersion" results, which means that some of the water may be treated much less and some much more than the nominal time. The time required for water to pass through a basin may be determined by a salt test. The data obtained may be plotted as a frequency curve. Using logarithmic probability paper it develops that the dispersion of the water in the basin follows closely the normal law of error. A quantitative measure of dispersion is developed, giving a numerical measure of efficiency of the basin as regards complete volumetric displacement.

Hazen's basic conclusion regarding plain sedimentation, that the capacity of a basin depends only on its surface area and is independent of its depth, does not apply to any basin where coagulation plays a part. It appears that volume is the true measure of sedimentation capacity, but this needs further experimental verification.

Twelve elements of sedimentation basin design are enumerated and the principles which should determine the decision regarding each element are discussed. Particular attention should be paid to inlet and outlet devices and to internal arrangements to decrease dispersion. "'Round-the-end baffles" are objectionable and unnecessary. Velocity is unimportant if it is less than 3 feet per minute. Covered basins are better but more expensive. Columns in basins should be avoided if possible.

DISCUSSION

JOHN B. DEAN:[7] Mr. Morrill's paper is very interesting and we all agree with him that information on this subject is very meager. His method of determining the "dispersion" and plotting the data is novel and his paper should stimulate us to make tests of this nature and add to the general fund of knowledge on the subject.

In 1926, the basins for our new plant at Howard Bend were designed for an initial capacity of 80 m.g.d. and our engineers, being confronted with the same lack of information on basin design were forced to design them largely by a process of simple reasoning, certain features of course being controlled by local conditions. It may be of interest to compare our designs with Mr. Morrill's recommendations.

Our layout consists of eleven basins, eight of which are 150 feet square with mechanism for continuous removal of sludge, five being

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intended for presedimentation and three for coagulation, but so arranged that, by moving stop planks, any number from two to six may be used for presedimentation and the remainder for coagulation. From these the water flows to three sedimentation basins each 404 feet by 330 feet. It will be seen that we have complied with Mr. Morrill's recommendations that there be a minimum number of three basins for each class of service.

In regard to the length and width, the smaller basins were made square to accommodate the Dorr mechanism, the larger ones slightly wider than they are long, the reasoning being that the velocity across the basin would be made less by making the width relatively greater and the length smaller. This is in accordance with the general practice of using basins in parallel, rather than in series in order to reduce the velocity. In this respect, I do not exactly agree with Mr. Morrill that the length should be three times the width, if proper provision is made against short-circuiting. The average depth of our smaller basins below normal water surface is 17 feet and of the larger basins 16 feet, but deducting 30 percent from the volume of the smaller basins and 15 percent from the volume of the larger ones for sludge storage the effective depths are 12 feet and 13½ feet respectively. The velocities and detention periods based on these effective volumes are as follows:

	VELOCITY	DETENTION TIME
	feet per minute	*hours*
Presedimentation basins	0.84	3
Coagulation basins	1.25	2
Sedimentation basins	0.45	12

These values are well within the limits recommended in the paper, viz., 3 feet per minute.

Water is delivered to and carried away from the basins by conduits approximately 8 feet by 10 feet in cross section which run the full length of each group of basins on the inlet and outlet sides, and is admitted to and drawn from the basins through 24 inch by 36 inch sluice gates located on the basin walls spaced 30 feet apart in the case of the small basins and 40 feet apart in the case of the larger ones. In order to avoid short circuiting a stilling baffle is built along the full length of the inlet and outlet side of each basin. It is

located 3½ feet from the wall and consists of precast slabs 16 inches wide by 9 feet long set up and concreted in place so that the slots between them are 2 inches wide on the side next to the gates and 6 inches wide on the side next to the center of the basin, the lower end of the slots being approximately 4 feet above the floor of the basin. This arrangement has apparently reduced short circuiting to a minimum and is very flexible from the standpoint of operation. By having the same arrangement on the inlet and outlet sides, the direction of flow may be reversed with very little effort and the baffle on the outlet side may be converted into a submerged weir by filling the lower portion of each slot with timber.

The velocities through the gates and open slots are as follows:

	THROUGH GATES	THROUGH SLOTS
	feet per minute	*feet per minute*
Presedimentation basins	50	9
Coagulation basins	83	16
Sedimentation basins	41	8

The sludge is removed from the presedimentation and coagulation basins by Dorr clarifiers. Up to date it has been removed from the sedimentation basins by draining and by washing with fire streams, but in the future it is proposed to use tractors equipped with bulldozers as is done at the Chain of Rocks Plant. The volume allowed for slugde in the sedimentation basins is sufficient for an accumulation of one year.

It may be interest to know that approximately 53 percent of the total solids are removed in the presedimentation basins, 33 percent in the coagulation basins, 13 percent in the sedimentation basins and 1 percent in the filter plant. The sludge from the coagulation basins is returned to the raw water entering the presedimentation basins and the water is conditioned in tangential mixers located between the presedimentation and coagulation basins.

JAMES W. ORTON:[8] Mr. Morrill's paper very adequately covers general features of sedimentation design. A large-sized book could be written on the subject going into details of design and experimentation. One point that he has not mentioned that was brought home

[8] Assistant Civil Engineer, Department of Water Supply, Detroit, Mich.

very forcefully in the model basin experiments at Detroit, is the subject of the sensitiveness of basins in general at low velocity flows.

In efforts to obtain good distribution of flow in a " 'round-the-end" baffle model, every possible device was employed including inlet walls with various sizes and shapes of openings, slotted walls with and without throttling arrangements, submerged weirs of varying depths, overflow weirs with varying levels, screens, A-frame baffles, training walls grouped at the inlet, training walls placed throughout the basin and many other schemes, besides various combinations of these devices. In all of these experiments the inflowing water was accurately controlled so as to make entrance conditions exactly similar, but in spite of this fact it was very noticeable that minute changes of basin control features produced very appreciable changes in the way the water was distributed. It was apparent also that a basin set-up that proved to be good at one particular flow would produce entirely different currents at another flow.

This was illustrated forcefully in the perforated inlet baffle when with 241 $\frac{1}{2}$-inch round holes, some of which were stopped off, the altering of one hole would appreciably change the basin action. In the training vane arrangements, the shifting in position of the vane nearest the inlet by as much as $\frac{1}{16}$ of an inch also entirely altered the basin flow. This method of training vanes was finally adopted and similar large sized vanes were installed in the basin; on the vane above mentioned, an adjustable sliding leaf was installed.

The function of the coagulation or settling basin is of extreme importance in the purification program. Too often it is carelessly designed, more thought being given to structural details than to flow conditions. By designing good inlet and outlet arrangements the basin can be made to carry its share of the load of purification. Often the basin is short-circuited with high velocities which permit of very little settling. This condition may not even be known to the operator who gives very little thought to the settling basin except at cleaning time.

Some time ago an engineer of note suggested that the sedimentation basin is of sufficient importance to warrant the services of an attendant. In large plants, subject to considerable fluctuations in daily and seasonal flow, it is believed that a regular attendant could very profitably be assigned to basin control duty, studying it in all its conditions of operation with perhaps adjustable inlet and outlet arrangements under his control. A more uniform applied water could thus be assured for the filters.

J. W. ELLMS:[9] The author of this paper discusses in considerable detail the factors affecting the design of sedimentation basins. His proposal to use a "dispersion index" to indicate the efficiency of a settling basin would have been more convincing if he had given some actual percentage removals of sediment by basins having different "indices of dispersion." The overflow rate as indicated by the ratio of flow in cubic feet per hour divided by the settling area, or the number of gallons flow per square foot per minute as determined by dividing the settling area of the basins by the flow in gallons per minute, also offer indices of value, provided their significance can be properly evaluated by sufficient data obtained from the actual performance of settling basins handling various types of water.

It is obvious that the physical characteristics of the suspended particles in various types of water, whether treated with a coagulant or not, may vary to a considerable degree. With the exception of those particles which may be so extremely small as to give to the water the properties of a colloidal suspension rather than a macroscopic suspension, there must be an unbalanced force tending to send all other particles to the bottom. The forces operating to prevent or retard deposition are those produced by the horizontal movement of the entire body of water, coupled with other cross currents and eddies induced by obstructions, such as columns, inlets, outlets, etc., and the viscosity effects due to temperature changes in the water. The reduction of these counteracting forces to the action of gravity is, therefore, the sine qua non of the best design for a settling basin.

In practical design the difficulty of forcing the water entering the basin to move at a uniform rate throughout the entire cross section of flow may be materially aided by the entrance slots and training walls provided in the new Springwells filter plant at Detroit. However, by carrying the inlet slots so close to the bottom of the settling basins, the question arises as to whether bottom velocities are not induced which may actually retard sedimentation. The writer can see no reason for regarding as a loss to basin capacity a certain proportion of the depth of a settling basin as a zone of practical quiescence, provided this depth is regarded as available for storage of sediment, and the upper cross section does not flow at velocities too

[9] Engineer of Water Purification, Department of Public Utilities, Cleveland, O.

high to prevent settlement within the period of time required for the passage of the water through the basin.

The agglomeration of coagulated particles during periods of sedimentation probably does change their subsiding values, but such a condition would seem to hasten rather than retard their deposition. There may still remain, however, particles so small that the longer period required for their deposition will offset the advantages obtained from the more rapid settlement of the agglomerated coagulated particles. Such complex conditions as these introduce difficulties in the practical design of settling basins, and lead naturally to conservatism in design.

While empiricism largely governs the design of sedimentation basins at the present time, the efforts of the author of this paper to place it upon a rational basis through model and large scale experiments are to be highly commended.

ABSTRACTS OF WATER WORKS LITERATURE[1]

FRANK HANNAN

Key: American Journal of Public Health, 12: 1, 16, January, 1922. The figure 12 refers to the volume, 1 to the number of the issue, and 16 to the page of the Journal.

Civil Engineering in Siam. HAROLD E. BABBITT. Eng. News-Rec., **106**: 928–31, 1931. Discussion of civil engineering in Siam and of municipal work in Bangkok. There is no comprehensive sewerage system in Bangkok. Water system, constructed in 1914 and added to in 1929–30, consists of a 53-million gallon reservoir formed by damming an old channel of the Menam Chao Phya 25 miles above city, an open canal to city, 4 coagulation basins, and 24 rapid sand filters of 17.5 million gallons per day total capacity. Alum is applied to raw water at rate of from 1.5 to 2.0 grains per gallon and Cl_2 to filtered water in amounts ranging from 2 to 3.5 p.p.m. Daily chemical and bacteriological examinations show the water to be much superior to U. S. Treasury Department standard. Recreational use of impounding reservoir is strictly prohibited.—*R. E. Thompson* (*Courtesy Chem. Abst.*).

Design and Construction of Two New Reclamation Dams. Eng. News-Rec., **106**: 998–1002, June 18, 1931. Illustrated description of design and construction of Gibson and Echo dams, completed by Bureau of Reclamation during 1930. Gibson dam, which forms reservoir 6 miles long storing 90,000 acre-feet of flood water for Sun River irrigation project, is of concrete arch type, with maximum height of 195 feet and crest length of 960 feet. Echo dam, of Salt Lake basin project, is of earthfill type and has maximum height of 125 feet above river bed and 151 feet above bedrock and 25-foot crest 1804 feet long.—*R. E. Thompson.*

Dependability of Water Service Requires Preparedness for Emergencies. Eng. News-Rec., **106**: 861–5, May 21, 1931. Emergency organizations and methods of following cities are described briefly: New York, Chicago, Detroit, Pittsburgh, New Orleans, St. Louis, and Peterborough.—*R. E. Thompson.*

Continuous Algae Control by Use of Chlorination. F. J. SETTE. Eng. News-Rec., **106**: 885–6, 1931. Chlorination of impounding reservoir at Pulaski, Va., by applying $Ca(OCl)_2$ to water leaking through rockfill dam before it

[1] Vacancies on the abstracting staff occur from time to time. Members desirous of coöperating in this work are earnestly requested to communicate with the chief abstractor, Frank Hannan, 285 Willow Avenue, Toronto 8, Ontario, Canada.

was pumped back into reservoir, effectively controlled algae growths and prevented development of tastes and odors usually experienced during summer months. Residual Cl_2 content of reservoir water varied up to 0.14 p.p.m. Tabulated data are given showing residual Cl_2 contents observed, total microscopic count, and number of *Dinobryon* present, latter being the predominant organism. Charlottesville reservoir water, which had a very pronounced fishy taste and odor in September, contained 615 standard units of *Dinobryon*, 48 of *Synura* and 18 of *Glenodinium*.—*R. E. Thompson* (*Courtesy Chem. Abst.*).

Design Limits on Providing for Peak-Load Consumption. ELMER G. MANAHAN. Eng. News-Rec., **106**: 856–7, May 21, 1931. Extent to which public will pay for facilities to take care of peak, or infrequent, demands is best index of how far to go in providing for such maximum loads. More liberal allowances for peak demands than have been warranted in past now appear justified, but it is not necessarily sound policy to provide for all peak loads, however great and whatever their cause. Unusual demands are due chiefly to four conditions: dry and hot weather; unusual growth of community in direction, character, and population; failure of mains or equipment; and large fires. The first two should be provided for fully in design of system. Ample reserve units should be installed and design should be such that breakdown, or failure, will incapacitate as little as possible of remainder of system. If system is accorded a fairly high rating under grading schedule of National Board of Fire Underwriters, peak demands, not only from fire, but also from most other causes, will have been provided for to as great an extent as is warranted. In case of fire demands, cost of additional facilities should be balanced against losses that might result from lack of such facilities.—*R. E. Thompson.*

Electric Eye Guards Water Supply of Denver. D. D. GROSS. Eng. News-Rec., **106**: 865–6, 1931. Most of Denver's water supply is diverted from South Platte River through intake located at South Platte Canyon, 23 miles distant. Although turbidity is usually comparatively low, sudden increases during spring flood periods frequently cause trouble. Recent experiments have shown that a photo-electric relay, arranged to sound a gong when turbidity increases, or decreases, greatly facilitates operation. Diagram of relay arrangement is shown.—*R. E. Thompson* (*Courtesy Chem. Abst.*).

Activities and Conditions at Boulder Dam. Eng. News-Rec., **106**: 895–7, May 28, 1931. General description of many activities in vicinity of Boulder dam site. First camp buildings are in course of erection at Boulder City, the permanent construction town, located 6 miles from dam. Town site was selected as area closest to dam of sufficient size to allow for anticipated development. Town water supply will be drawn from Colorado River downstream from dam site, allowed to settle, pumped about 6 miles against head of 2,000 feet to storage tank, and passed through filter plant to distribution system.—*R. E. Thompson.*

Buenos Aires Subway 4.6 Miles Long Built in Twenty Months. Eng. News-Rec., **106**: 923–6, June 4, 1931. Illustrated description of construction of first

section of 8.7-kilometer subway of Ferrocarril Terminal Central de Buenos Aires. About half the completed section was built in open cut and remainder in tunnel, and all of second section is being built in tunnel.—*R. E. Thompson.*

Covered Concrete Reservoir at Springfield, Massachusetts. Eng. News-Rec., **106**: 912, May 26, 1931. Data given from bids on extension of covered masonry reservoir of Springfield water system. New basin will be 288 by 224 feet in area and will have capacity of 11 million gallons. Contract was let for $171,469, the lowest bid. Average of 20 bids was $253,016 and highest $352,260. —*R. E. Thompson.*

Acidity and Hardness Difficulties at Monongahela River Plants. L. S. MORGAN. Eng. News-Rec., **106**: 850, 1931. Outline of problems encountered during recent drought at plants treating Monongahela River water, due to drainage from active and abandoned coal mines and, to a lesser extent, to "acid iron" wastes from industrial works. Alkalinities as low as −142 p.p.m. and hardnesses as high as 540 p.p.m. were recorded. At one plant hardness averaged 400 p.p.m. for 6 months and as much as 27 tons of Na_2CO_3 and 12 tons of lime were required per day to soften 14.3 million gallons of water. Studies are being made of the mine drainage problem.—*R. E. Thompson* (*Courtesy Chem. Abst.*).

A Service Building for Public Utilities. THEODORE A. LEISEN. Eng. News-Rec., **106**: 858–60, May 21, 1931. Description of new service building of Metropolitan Utilities District of Omaha, Nebraska, serving water, gas, and ice departments. Ample and well-planned accomodation is provided for storage of materials, meter repairs, automobile storage and repair, shop office, and complete apartment for emergency crew. Adjacent to building is yard for storage of cast iron pipe, large valves, and other cumbersome equipment. —*R. E. Thompson.*

Principles of Financing the Privately Owned Water Works. V. BERNARD SIEMS. Eng. News-Rec., **106**: 848–9, May 21, 1931. Outline of subject, in which it is pointed out that budgetary control of operating and capital expenditures is necessary to ensure efficient management. Intelligently used, reserve funds are very important to any budgetary control system, as they tend to flatten out peaks and enable operations to be controlled more easily. Relation of maintenance to depreciation and of these two to replacements is discussed briefly.—*R. E. Thompson.*

Colloid Chemical Questions of Feed Water with Particular Consideration of Desilicifying. R. STUMPER. Die Wärme, **53**: 859–62, 1930. From Chem. Abst., **25**: 1013, March 10, 1931. Colloids have recently gained importance in boiler operation in connection with such problems as silicate boiler scale, foaming, and corrosion. Foaming and effect of colloidal material on origin of precipitates in boilers have been theoretically treated by LANGMUIR (cf. C. A., **22**: 470). Practical application of this work is discussed. Stabilizing of the colloidal state may be of use in prevention of boiler scale, but it may also

bring about foaming. Application of colloidal boiler scale inhibitors must be tested under operating conditions. Importance of consideration of silicic acid and of characteristics of silicate boiler scale formation are discussed. Three processes are suggested for desilicifying: (1) condensation of silicic acid to colloidal silicic acid, (2) precipitation of silicic acid as an insoluble silicate, and (3) a combined chemical and colloidal process. Through protective action of known colloids, e.g., humus materials, precipitation of scale in water softener and boiler can be retarded, while precipitated material will be kept in colloidal condition.—*R. E. Thompson.*

Of What Importance is Sodium Phosphate in Boiler Feed Water Treatment? HAENDELER. Die Wärme, **53**: 881–3, 1930. From Chem. Abst., **25**: 1014, March 10, 1931. Effect of sodium phosphate in feed water is three-fold; it removes residual hardness, inhibits corrosion, and removes scale previously formed. Well treated feed water will have residual hardness of from 0.25 to 2.0° [4.5 to 35 p.p.m.], which increases by evaporation. Excess soda precipitates residual hardness as loose carbonate sludge in boilers operating at pressures under 15 atmospheres. Above that pressure scale is formed, as high temperature decomposes the soda. Sodium phosphate, in contrast, is stable and forms a difficultly soluble and finely divided alkaline earth phosphate. The higher cost of sodium phosphate will prohibit its use where cheaper reagents produce equal residual hardness. Presence of sodium phosphate produces definite protective action against corrosion. Effect of sodium phosphate is 500-fold greater than that of equal amount of sulfate and it may, therefore, be used in smaller concentrations. The lower salt content will improve quality of steam by decreasing foaming and knocking (cf. C. A., **23**: 3287). Small excess of phosphate will dissolve scale previously formed, this action being due to a slow base-exchanging process. Analytical methods for determining phosphate offer difficulties. Colorimetric methods are described. Concentration of sodium phosphate in feed water should be maintained at from 15 to 30 p.p.m., expressed as PO_4.—*R. E. Thompson.*

Determination of Phosphates in Presence of Silica in Boiler Water. ELWOOD W. SCARRITT. Ind. Eng. Chem., Anal. Ed., **3**: 23, 1931. From Chem. Abst., **25**: 1014, March 10, 1931. Presence of silicic acid causes development of blue color similar to that produced by phosphoric acid in colorimetric determination with molybdate. By properly adjusting the H-ion concentration, however, the error may be overcome; but standards must be adjusted to same pH value as sample. Directions are given in detail.—*R. E. Thompson.*

Removal of Dissolved Gases from Boiler Water for Prevention of Corrosion. W. A. S. HARMON. Petroleum World and Oil Age, **27**: 90–3, 95, 118, 120, 1930. From Chem. Abst., **25**: 1014, March 10, 1931. Deaërators for reducing corrosion are of following types. (1) Heated water is admitted to chamber under vacuum so that it boils, or flashes, throwing off dissolved gases. This deaërator, known as the flash type, is particularly effective for removing ammonia and hydrogen sulfide. (2) Heated water is injected into vessel through which steam at same temperature as water is sweeping. (3) Heated water is

rained over evaporating tubes carrying steam. (4) Water cascades over series of heating trays, then over series of deaërating trays. Steam atmosphere in deaërating section is gas-free, hence films of water continuously give off their dissolved gases to it. In severely corrosive waters, adjustment of pH value to 9.5 after deaëration will inhibit corrosion.—*R. E. Thompson.*

The Drinking Water Problem in Nanking. G. L. KUO. Chem. Ind. (China) (2), **5**: 58–67, 1930. From Chem. Abst., **25**: 1012, March 10, 1931. Nanking has no modern water supply system. Analyses indicate that river water is preferable to well water. Improved methods are suggested for city-wide transportation of water for domestic use.—*R. E. Thompson.*

Sanitation and Potable Water for China. A. M. SHAW and J. A. L. WADDELL. Far Eastern Rev., **25**: 396–7, 1929. From Chem. Abst., **25**: 1915, March 10, 1931. *R. E. Thompson.*

The Storage of Activated Charcoal. JOSEF KADLEC. Listy Cukrovar., **49**: 169–70, 1930. From Chem. Abst., **25**: 1040, March 10, 1931. Brick storehouses with brick flooring are fitted with layer of beams covered with planks, ventilation beneath bin being thus permitted. Activated charcoal is placed in bins in 20-centimeter layers. Moisture content changes continually with that of atmosphere of storehouse. Turning over once every month is recommended. Bagging is not recommended; acidity of charcoal disintegrates bags. Greenish white moldy growth occurs on old material; it is caused by *Penicillium glaucum.* Fermentation leading to spontaneous combustion has also occurred. Moldy appearance is sometimes due to crystalline sodium chloride and occurs with incomplete washing charcoal. Activated charcoals show no loss of effectiveness up to the temperature of 280°.—*R. E. Thompson.*

Laundry Experiments. W. KIND. Seifensieder-Ztg., **57**: 888–9, 1930. From Chem. Abst., **25**: 1112, March 10, 1931. Author points out importance of hardness of water in washing tests, not only in soap solution proper, but also in rinsing; insoluble soaps are formed in both cases.—*R. E. Thompson.*

The Treatment of Water for Bleaching and Dyeing. C. D. BLACKWELDER. Am. Dyestuff Reptr., **20**: 29–31, 1931; cf. C. A., **25**: 597. From Chem. Abst., **25**: 1088, March 10, 1931. Discussion of difficulties caused by unsuitable water.—*R. E. Thompson.*

The Radioactivity of Stone Mountain Springs. JAMES A. HOOTMAN and W. S. NELMS. Am. J. Sci. (5), **21**: 37–8, 1931. From Chem. Abst., **25**: 1152, March 20, 1931. Electroscopic measurement of radium emanation from 14 samples of water from springs around Stone Mt. showed 0.40 to 15.60, average 4.2, millimicrocuries per liter.—*R. E. Thompson.*

Determination of Oxygen Dissolved in Sea Water. MAURICE NICLOUX. Bull. Inst. Oceanographique, **563**: 19 pp., 1930; cf. C. A., **24**: 4999. From Chem. Abst., **25**: 1179, March 20, 1931. Microdetermination of oxygen in sea

water with 5 to 25 cc. samples and special reaction tube is described in detail. Manganic hydroxide formed is treated with acid and potassium iodide and liberated iodine titrated with sodium thiosulfate.—*R. E. Thompson.*

Spot Test for Nitrite. FRIEDRICH L. HAHN. Mikrochemie, **3**: 31–3, 1931. From Chem. Abst., **25**: 1181, March 20, 1931. Directions are given for preparing pure α-naphthylamine oxalate and for cleaning, moistening, and drying filter paper in suitable manner for testing. When proper precautions are taken, test for nitrite is very sensitive.—*R. E. Thompson.*

Corrosion and Metal Protection in Steam Power Plants. III. Examination of a Cracked Condenser Tube. R. STUMPER. Korrosion und Metallschutz, **6**: 217–8, 1930; cf. C. A., 24, 3299. From Chem. Abst., **25**: 1206, March 20, 1931. Examination of cracked brass condenser tube led to conclusion that cracking was due to severe internal stresses of cold-drawn tube and not to any special form of corrosion. In order to prevent cracking, tube should have been annealed after cold drawing, or if greater hardness was desired, it should have been heated to 250–300°.—*R. E. Thompson.*

Corrosion of Metals. F. MÜLLER. Z. angew. Chem., **43**: 225–9, 1930. From Chem. Abst., **25**: 1206, March 20, 1931. Review of recent theories and experiments.—*R. E. Thompson.*

Action of Oxygen on the Corrosion of Iron and Steel in Aqueous Solution. HIKOZÔ ENDO and SHIGENORI KANAZAWA. Sci. Rept. Tôhoku Imp. Univ. 1st Ser., **19**: 425–35, 1930. From Chem. Abst., **25**: 1206, March, 20, 1931. Experiments were carried out to investigate action of dissolved oxygen upon corrosion of iron and steel. Potential measurements confirmed theory of oxygen concentration cells. Corrosion in aqueous solutions is considered to take place in 2 stages, and dissolved oxygen is thought to have accelerating effect on both. Primary action of dissolved oxygen lies in formation of concentration cells and ensuing acceleration of solution of iron, forming solution of ferrous iron. Secondary action concerns removal of hydrogen film and oxidation of ferrous hydroxide. Various phenomena of corrosion are explained by considering these two separate factors.—*R. E. Thompson.*

The Corrosion of Metals. G. L'HOIR. Bull. assoc. étudiants école sup. brasserie univ. Louvain, **30**: 101–6, 1930. From Chem. Abst., **25**: 1206, March 20, 1931. Brief review of electrolytic theory of corrosion and of factors facilitating corrosion.—*R. E. Thompson.*

Fundamentals of Corrosion. G. MASING. Z. Metallkunde, **22**: 321–6, 1930. From Chem. Abst., **25**: 1205, March 20, 1931. Review of theories. *R. E. Thompson.*

Significance of Burning Depth in Arc Welding and Means for Its Improvement. ERNST SCHWARZ. Z. Ver. deut. Ing., **74**: 1565–7, 1930. From Chem. Abst., **25**: 1208, March 20, 1931. Welding wires with low carbon content give

greater burning depth. Diminishing velocity of feed and increasing welding current also increase burning depth within certain limits. Following recommendations are made regarding most favorable burning depth for various types of welds: for butt welds, 2 mm.; for V-welds, depth must be at least sufficient to avoid hollow space in vertex of angle; for cast iron welds, depth should be greatest attainable.—*R. E. Thompson.*

Fatigue Strength of Welded Steel Joints. W. HOFFMANN. Z. Ver. deut. Ing., **74**: 1561–4, 1930. From Chem. Abst., **25**: 1208, March 20, 1931. Welded pipe joints have average vibratory strength about 50 per cent of that of material. Strength is influenced by width and thickness of weld as well as by ease with which material is welded. Nature of fluxing material is also of importance. Red-short welds show little fatigue resistance. In general, burning depth should be as deep as possible and weld should have flat surface. Both impact and vibratory fatigue strength are higher for welded than for riveted joints.—*R. E. Thompson.*

Technical Safety Questions in Acetylene Welding. EDUARD SAUERBREI. Z. Ver. deut. Ing., **74**: 1591–5, 1930. From Chem. Abst., **25**: 1208, March 20, 1931. Discussion of various causes of explosions in acetylene welding, including formation of detonating gas and possibility of igniting explosive mixtures. Air penetrates into equipment in number of ways; during removal of sludge, renewal of water, etc. Ignition of acetylene-air mixtures is brought about by spark formation, incandescent carbide, open flames, striking back of flame, sucking back of oxygen during welding, and handling frozen equipment.—*R. E. Thompson.*

Thermotechnical Measuring Arrangement of a Boiler House in the Plan of an Industrial Power Plant. A. GRÜNWALD. Die Wärme, **53**: 923–6, 1930. From Chem. Abst., **25**: 1300, March 20, 1931. Control panels and control instruments (pyrometers, flow meters, and flue-gas analyzers) are discussed in regard to their design, selection, installation, and arrangement for modern boiler plant.—*R. E. Thompson.*

Microdetermination of Potassium in Waters. H. GRIFFON and Mlle. A. BERNARD. J. pharm. chim. (8), **12**: 118–24, 1930. From Chem. Abst., **25**: 1309, March 20, 1931. Micro- method used is that of LEULIER, VELLUZ, and GRIFFON (C. A., **22**: 4405; **24**, 3549). Results of examination of 24 samples of French waters, at least two determinations on each, are tabulated. Results for each sample agree well with one another, but differ from other recorded results; e.g., for a certain water, BARRAL (C.A., **24**: 907) found 2.15 p.p.m.; an old record gives 1.64; authors found 3.77 and 3.79.—*R. E. Thompson.*

Concerning the Control and the Degree of Reliability of the Chlorinating Process of Purifying Drinking Water, Especially in Relation to the Use of Chloramine for This Purpose and the Chlorine Process as Applied to Water Containing Ammonia in Solution. II. K. HOLWERDA. Mededeel. Dienst Volksgezondheid Nederland-Indië, **19**: Pt. 2, 325–85, 1930; cf. C. A., **23**: 1704. From Chem.

Abst., 25: 1306, March 20, 1931. In continuation, author studied chemical action of hypochlorites on water containing ammonia by varying chlorine: ammonia ratio at low concentrations and at pH 6.5 to 6.8. It was found that when ratio was lower than 6.3:1 (p.p.m.) only monochloramine with traces of dichloramine and nitrogen trichloride remains; but, with higher ratios, hypochlorous acid persists, in addition to increasing amounts of nitrogen trichloride as ratio increases. During course of work, methods were devised for determining nitrogen trichloride, dichloramine, monochloramine, and hypochlorous acid in mixtures of all four. Comparison was also made of different methods of determining active chlorine. It was found that manganese and iron do not seriously interfere with *o*-tolidin test, but that 2 p.p.m. nitrite (NO_2) will give color comparable to 0.5 p.p.m. chlorine after 5-minute contact with *o*-tolidin. It is recommended that both *o*-tolidin and methyl orange be used to determine active chlorine. Author found that in ammonia-free water of pH 6.8, coli count is reduced from 310 to 0 in 1 minute when 0.1 p.p.m. chlorine is added; but that when 0.03 p.p.m. ammonia (NH_3) is present, count is only reduced from 310 to 100 in 20 minutes. Further experiments showed that in presence of ammonia there was marked disinfectant action due to nitrogen trichloride and dichloramine. Author concludes that *o*-tolidin test does not always give reliable value on which conclusions may be based regarding reliability of chlorination.—*R. E. Thompson.*

The New Venturi Meter Installations in the Munich Water System. E. HENLE. Gas-u. Wasserfach, 73: 1225–32, 1930. From Chem. Abst., 25: 1304, March 20, 1931. Venturi meters have been installed on various water sources and overflows, permitting check on consumption at any given time. New Venturi meters have been designed which operate on much lower head than those used heretofore.—*R. E. Thompson.*

Research into the Properties of the Aërobe, the Non-Sporulating Lactose Fermenting Bacteria in Feces and Soil in the Tropics and the Importance Thereof for the Analysis of Drinking Water. K. HOLWERDA. Mededeel. Dienst Volksgezondheid Nederland.-Indië; 19: Pt. 2, 289-324, 1930. From Chem. Abst., 25: 1307, March 20, 1931. Survey was made of types of coli-like bacteria in native feces and in soil in Dutch East Indies. Organisms were isolated from feces by planting fresh suspension on lactose-peptone-agar (containing 0.5 per cent sodium taurocholate). Strains from soil were obtained by preliminary enrichment in lactose broth (sometimes containing 0.5 per cent sodium taurocholate). Of 674 strains from feces, 24 were V.-P. (VOGES PROSKAUER) + and 650 V.-P. −. Of the 650 V.-P. − cultures only 1 was citrate +, while 649 were citrate −. Of 953 soil cultures, 672 were V.-P. + and 279 V.-P. −. Of the latter, 151 were citrate + and 128 citrate −. Contrary to other workers, author finds higher percentage of aërogenes in habitated soils than in soils from uninhabited places. He concludes that both quantitative evaluation of number of coli-like organisms and their differentiation should be considered in bacterial analysis of water.—*R. E. Thompson.*

Nitrate Determinations in Drinking Water. K. SCHERINGA. Pharm. Weekblad, 67: 1362–3, 1930. From Chem. Abst., 25: 1309, March 20, 1931.

To sample containing approximately 0.1 milligram NO_3 add 1 cc. 0.5 per cent sodium salicylate and evaporate to dryness on water bath. Cool, add 1 cc. concentrated sulfuric acid and allow to stand 10 minutes. Rinse with water into colorimeter glass, add 10 cc. of sodium hydroxide or ammonium hydroxide and compare with standard. If much chloride is present multiply by factor $100/(100 - 10a)$, in which a represents milligrams Cl.—*R. E. Thompson.*

Protection of Concrete with Coatings Containing Metallic Aggregates. GEORGE DUPONT. Ciment, **35**: 462–74, 1930. From Chem. Abst., **25**: 1355, March 20, 1931. Mortars made of metallic aggregates and cement are useful in providing durable and impervious coatings for concrete exposed to sea water, or other aggressive solutions, to frost action, or to erosion by running water.—*R. E. Thompson.*

The Determination of Manganese in Potable Waters. EDWARD BARTOW and HARRIS THOMPSON. Proc. Iowa Acad. Sci., **36**: 245–50, 1929. From Chem. Abst., **25**: 1309, March 20, 1931. Manganese was determined in 12 samples of water by sodium bismuthate, ammonium persulfate, and potassium periodate methods. No relationship was found between presence of manganese and alkalinity, chloride, and iron contents. Periodate method is fully as accurate as the 2 methods in general use at present time. It is much shorter than persulfate method because all chlorides are oxidized to chlorine gas which is removed in evaporation; thus precipitation of chlorides with silver nitrate is unnecessary. It is shorter than sodium bismuthate because excess of potassium periodate is colorless and soluble in the acid solution and need not be removed before comparing with color standards. True permanganate color was always obtained in periodate oxidation. Cost of potassium periodate is small, only 0.3 gram being required for each analysis. No special apparatus is required.—*R. E. Thompson.*

Waste Waters from Sugar Factories. C. MANUELLI and S. ANSELMI. Atti III congresso naz. chim. pura applicata, **1930**: 607–26. From Chem. Abst., **25**: 1314, March 20, 1931. Disposal of sugar factory effluents is very important in Italy because of lack of deep wells for domestic use, and also particularly difficult because factories are massed closely together. Total water used varies from 6000 to 30,000 cubic meters per 24 hours per factory. Most of this is used for washing and transporting beets and is relatively innocuous. Diffusion, press, and pulp waters are more difficult to deal with. Previous literature is reviewed and brief critical description of mechanical, chemical, and biological disposal methods is given. Repeated use of water and sprinkling of large areas with waste waters are also discussed. Combinations of these methods are now used successfully in Italy, exact method depending on local conditions.—*R. E. Thompson.*

The Nürnberg Water Supply. FR. KRAUSS. Gas- u. Wasserfach, **73**: 1156–9, 1930. From Chem. Abst., **25**: 1304, March 20, 1931. Measures to increase water supply are discussed.—*R. E. Thompson.*

Comparison of the Dominick-Lauter Test for B. Coli in Water with that of Standard Methods. Final Report. HAROLD W. LEAHY, J. W. FREEMAN and CHRIS P. KATSAMPES. Am. J. Public Health, 21: 11–8, 1931; cf. C. A., 25: 368. From Chem. Abst., 25: 1307, March 20, 1931. Samples of water, numbering 1116, from 358 different sources, have been tested according to procedure of Standard Methods and, in comparison, with new methylene-blue bromocresol purple medium of DOMINICK and LAUTER (cf. C. A., 23: 4986). Percentage of tubes showing gas which confirmed and total number of samples showing gas were much higher with new medium than with standard lactose broth. Percentage confirmation for standard lactose broth tubes showing gas in 24 hours was higher than for tubes showing gas in 48 hours, but percentage confirmation with DOMINICK-LAUTER tubes showing gas in 24 and 48 hours was approximately same in each case. Percentage confirmation for lactose broth increased as pollution of water under examination increased, whereas percentage confirmation for new medium remained approximately the same. Of total number of samples showing positive presumptive test, 100 percent of those from new medium confirmed while only 68.7 percent from standard lactose broth confirmed. New DOMINICK-LAUTER medium was found to be far superior to standard lactose broth for detection of *B. coli* in water.—*R. E. Thompson.*

High Pressure Pays. M. K. DREWRY. Power Plant Engineering, 35: 249–50, 1931. From Chem. Abst., 25: 1310, March 20, 1931. Corrosion is considered more serious problem than scale formation. Many methods have been worked out for control and elimination of oxygen, which is agent most responsible—*R. E. Thompson.*

Recent Instances of Embrittlement in Steam Boilers. FREDERICK G. STRAUB. Trans. A. S. M. E. 52: 27, 339–45, 1930. From Chem. Abst., 25: 1310, March 20, 1931. Conditions observed in recent cases of boiler embrittlement are reported. Boiler failure at Crossett, Arkansas, is attributed to embrittlement resulting from use of sodium carbonate treatment of water too low in sulfate content. Emphasis is placed upon necessity for regular inspection of leaky seams in boilers, particularly when boiler water does not meet A. S. M. E. specifications.—*R. E. Thompson.*

Embrittlement in Boilers. FREDERICK G. STRAUB. Univ. Ill. Eng. Expt. Sta., Bull. 216: 125 pp., 1930. From Chem Abst., 25: 1311, March 20, 1931. This report is partial reprint of Bulletins 155 and 177 bearing on subject of embrittlement of boiler plate (C. A. 20: 2814; 22: 3125), together with new material dealing chiefly with solubilities of embrittling inhibitors in boiler waters, and with concentration of solutions in capillary spaces. General conclusions follow. (1) Embrittlement in boiler plate is caused by combined action of stress and chemical attack: stresses are inherent in construction and operation of boiler, while chemical attack is caused by presence of sodium hydroxide in boiler water. (2) Certain methods of water treatment tend to render dangerous some waters originally safe; e.g., treatment of waters low in sulfate with soda ash. (3) Laboratory tests with special apparatus have

shown that concentrations of sodium chloride up to 150 grams per liter may be built up in capillaries from concentration in large connecting tube of only 2 grams per liter. (4) Presence of sodium sulfate retards embrittling action and, if in proper proportion, prevents it entirely. (5) Presence of phosphate, tannate, chromate, nitrate, or acetate, also inhibits embrittling action of sodium hydroxide; some of these are used in large power plants. (6) No steel suitable for boiler plate has been found which is resistant to embrittling action of caustic soda.—*R. E. Thompson.*

Effect of the Mixing Water on Mortar Strength. E. PROBST and K. E. DORSCH. Zement, **19**: 1009–11, 1930. From Chem. Abst., **25**: 1354, March 20, 1931. General survey of concrete construction and strength in principal cities of Germany showed considerable differences in latter, apparently due to chemical differences in mixing waters used. Two ordinary portland cements, a high-early-strength cement, a slag cement, and an aluminous cement were compared in mortars mixed (*a*) with distilled water and (*b*) with Karlsruhe water having average hardness equivalent to 150 p.p.m. calcium oxide [270 p.p.m. calcium carbonate]. It was found that distilled water gave roughly 10 percent higher tensile strength, but considerably lower compressive strength in general than did Karlsruhe water.—*R. E. Thompson.*

The Relation between Corrosion and Paint. W. H. J. VERNON and L. A. JORDAN. Chemistry and Industry, **50**: 52, 1050–1057, December 25, 1931. Corrosion is grouped into three types; (1) where confined to exposed surface, (2) where some intercrystalline penetration, and (3) where changes extend through material (graphitization). Natural protective films, passivifying agents, anodic oxidation, copper steels and protection of magnesium with selenium are compared with paint films. Lateral continuity and impermeability are essentials in paints and also inhibition of corrosion by pigments. Lead oxide inhibits corrosion of iron; iron oxide is inert; while graphite, lampblack, and some metallic pigments stimulate corrosion under some conditions. Electrolytes must not be present under paint film and, to obviate this, it is often well for brush to follow blow-lamp treatment.—*W. G. Carey.*

Fort Canning Covered Service Reservoir, Singapore. E. F. REID. Water and Water Engineering, **33**: 398, 655–671, Special Winter Number 1931. Read before Institution of Water Engineers, December 4, 1931. Detailed illustrated description of 30-m.g. (Imp.) reservoir built on hill, 140 feet above sea-level, in centre of city. Lowest bid for contract was £268,730; but it was actually built by administration for £173,400 in 21 months. Reservoir is oblong, with concrete division-wall 10 feet high; height is 30 feet, of which 10 feet is below ground. Floor is plain concrete, 12 inches thick, covered with half an inch of asphalt. Honeycomb system roof consists of 264 plain concrete domes, hexagonal in plan, each resting on six reinforced groin arches springing from hexagonal columns. Columns, three feet thick, are built of hollow pre-cast concrete blocks weighing about half ton. These were used to save time, being made during excavation, and by using rapid hardening cement two sets were cast from one mould per day, the hot, moist climate facilitating rapid

hardening. Each block has hole 15 inches diameter which was filled with concrete after block set in position. Side walls are series of 64 horizontal arches of 30 feet span, sprung from buttresses 4 feet thick, the whole being surrounded by earth embankments. Buttresses act as vertical cantilevers and horizontal arches are reinforced circumferentially with pure tension bars hooked into buttresses. Scale model (one eighth) of complete buttress reinforcement was used to explain to native contractor fitting of the various rods. Time allowances for each operation were carefully calculated and progress charts on squared paper, with ideal theoretical rate as datum, were of greatest possible value in showing true rate of progress.—*W. G. Carey.*

Repairs to an Old Earth Embankment at Macclesfield, England. G. W. COVER. Water and Water Engineering, **33**: 398, 672–679, Special Winter Number 1931. Read before Institution of Water Engineers, December 4, 1931. Owing to slip in outer slope of embankment of 80-year old, 30-m.g. (Imp.) reservoir, excavations were made and boreholes sunk to determine underlying material and depth of rock. Four fractures in 12-inch cast iron pipes were considered to be due to sagging of embankment causing spreading of toes thus putting pipes in tension. Repairs included culvert of pressed steel surrounded with concrete to contain new outlet main, raising of puddle core and top of embankment to provide safe freeboard, drainage of outer slope to counteract tendency to slip, and cement grouting of millstone grit rock beneath seat of embankment.—*W. G. Carey.*

The Transmission of Disease by Water. A. W. BURTT. Water and Water Engineering, **33**: 398, 679–689, Special Winter Number 1931. Read before Institution of Water Engineers, December 4, 1931. In highly developed countries water-borne diseases are well under control; but in tropical countries, impure water is an important factor in spread of disease. Bacteria responsible for enteric fevers, dysentery and diarrhoea, Asiatic cholera, infective jaundice, and goitre are discussed, as well as diseases caused by protozoa and helminths, or parasitic worms. In almost all water-borne diseases, primary source of infection is excreta of man, or of animals, and although detection of specific bacteria of diseases mentioned is of great technical difficulty, *B. coli* tests with differentiation of *B. lactis aerogenes* are widely adopted. Any purification process delivering water free from *B. coli* in 50 or 100 cubic centimetres eliminates any pathogenic bacteria.—*W. G. Carey.*

Sterilization of Drinking Water by the Carbochlorine Process. La Nature, November 15, 1931, 452–454. From Water and Water Engineering, **34**: 399, 42, January 20, 1932. Clarified water is treated with sodium hypochlorite in the proportion of 0.0015 gram of available chlorine per litre of water and is then run through a tank containing specially prepared carbon. Combination of hypochlorite and active carbon gives a clear, aërated water, free from odour, flavor, and germs. Dr. CAMBIER of the Paris Laboratory of Hygiene has constructed plant to deal with large volumes of water.—*W. G. Carey.*

Composition of Water Demand of Large Cities. Anon. Water and Water Engineering, **34**: 399, 2, January 20, 1932. Analysis of different uses of water

in a city is important in calculating future requirements and planning extensions. In Vienna, water is metered in such a way that it is possible to classify demand according to domestic, industrial, public uses, loss through leakages and defective meters, etc. Analysis of amounts used in districts of different types and at different times of year show that increased summer demand is practically entirely due to bathing places, ice factories, gardening, holiday traffic on railways, hotels, etc.—*W. G. Carey.*

Artesian Water under London. Anon. Water and Water Engineering, **34:** 399, 2, January 20, 1932. Large business firms are finding artesian wells on premises running dry, owing to lowering of water table under London. Some users work pumps day and night to supply sufficient water. At present water level is from 200 to 500 feet below surface and is sinking in some instances 4 feet a year. Pollution by infiltration of salt water from Thames is taking place.—*W. G. Carey.*

Sterilization of Wells. Anon. Water and Water Engineering, **34:** 399, 3, January 20, 1932. Chlorination of new wells is essential, when well supplies municipality. During construction, contamination may be introduced by drilling tools, or by polluted gravel. Wells may be chlorinated under pressure, or by introducing strong hypochlorite solution; former is more effective, but requires special pumping equipment.—*W. G. Carey.*

The Action of Bacteria in the Tuberculation of Water Mains. F. W. Hammond and A. Goffey. Water and Water Engineering, **34:** 399, 7–8, January 20, 1932. Following the work by H. G. Ruddick and S. E. Linderman in America, authors give their experiences of tuberculation in England. Soft acid waters cause tuberculation and iron bacteria are always associated therewith. Maximum activity of crenothrix is at pH 5.8–6.4; of gallionella, at 6.4–6.8; of cladothrix, at 7.0–7.2; while leptothrix and spirophylla are common from pH 5.8 to 7.8. For growth of definite tubercles of ferric hydroxide, presence of exposed iron is essential, smallest pinhole in protective coating being sufficient. Long streamers, or slime, only, are found on inert substances, e.g. wood; but these are of bacterial origin. Concrete-lined pipes certainly give alkaline character to acid water which inhibits bacterial activity; but disintegrating effect upon concrete is present and relative brittleness and liability to hair cracks during setting and maturing raise doubts as to durability. Steel tubes with spun bitumen lining are resistant to tuberculation.—*W. G. Carey.*

The Ternary Alloys of Lead. Water and Water Engineering, **34:** 399, 3–4, January 20, 1932. Alloys possessing mechanical strength and resistance to fatigue far higher than lead, while retaining fine working properties of lead, have been studied by British Non-Ferrous Research Association and by special committee of British Waterworks Association. Lead-cadmium-autimony and lead-cadmium-tin are best and light weight test pipes of latter (98.25, 0.25, 1.50 percent) were filled with water and subjected to high pressure (1500 pounds per square inch) together with medium pressure lead pipes. Lead pipe failed in every case and, although alloy pipe had slightly increased in external

diameter owing to expansion, there was no indication of bulging or other evidence that bursting pressure had been nearly reached.—*W. G. Carey.*

Control of Water in Large Volumes and under High Velocity of Discharge through Dams and Pressure Tunnels. E. B. BALL. Water and Water Engineering, **34**: 399, 27–30, January 20, 1932. As result of high head in modern dams, special problems have arisen, such as turbulence, shock, and vibration, especially in outlet culverts having upstream regulation, and destructive energy released from downstream controlled outlets. Erosion of culvert linings is dealt with and the free vortex principle for avoiding destruction caused by energy contained in high head hydraulic jets is detailed. It is of primary importance to provide isolating equipment giving efficient emergency closing to operating sluices, or valves, and various types of valve and emergency closing gates are described. Design and construction of mechanical devices for controlling high pressure flowing water is highly specialised work. —*W. G. Carey.*

The Critical Depth of Flow in Open Channels. G. S. COLEMAN. Published by Institution of Civil Engineers, Westminster, London. From Water and Water Engineering, **34**: 399, 43, January 20, 1932. Abstract of paper (4791) read before Institute. Critical depth is new factor in solution of non-uniform flow problems, introduced in 1916 by KENNISON. Theoretically it occurs at fixed position of channel and, if measured in existing channels, rate of discharge can be ascertained, being expressed as function of critical depth. Experiments described in paper were (1) to find position and prove value of critical depth in open channels of circular cross section when discharging freely and (2) to ascertain relation between theoretical and experimental values for profiles for free surfaces of flow. It was found that critical depth bore constant ratio to recorded depth at free outlet for all gradients below 1 in 200 and that profiles agreed with theory. It is considered that theory can be utilised within limits specified.—*W. G. Carey.*

Shanghai Waterworks Extensions. H. STRINGER. The Engineer, **152**: 3962, 648–650. December 18, 1931 and 3963, 669–671, December 25, 1931. International Settlement, comprising about one million people, is supplied with water from Huangpu river. With turbidity of 223 and bacterial content of 50,000 at 37°C., this is one of most turbid and polluted water supplies in world and daily demand has risen from 11.5 gallons (Imp.) per head in 1905 to 44 in 1930. Additional outshore intake of 3 m.g.d. capacity was constructed in 1925, and further 5.5-m.g.d. inshore intake in 1929. Water is pumped to settling tanks, then goes to rapid filters, and is chlorinated before delivery to mains. Success of continuous settlement tanks depends on adequate alum mixing, accurate regulation of water velocities, and correct slope to mud pockets ensuring continuous flow of deposit through pipes to mud pumps. New settlement tank has 23 channels, triangular in section with 45° slope at right angles to direction of flow. Water has turbidity of 30 when delivered to rapid filters operating at 100 gallons per square foot per hour [125 m.g.a.d. U. S.], two batteries of eight filters each constructed since 1925, each having

capacity of one million gallons per hour. Under-drainage and backwash system is "Illinois Standard;" experiments with porous concrete system were unsuccessful. Filters are washed at 6-foot loss of head, but always once in 24 hours, percentage of wash water being 3.62 of total. Bacterial reduction from river to filter effluent is 99.51 percent and, to chlorinated effluent, 99.99 percent. Prechlorination was introduced in 1929 and has reduced bacterial load on filters by 90 percent, average dose being 0.5 parts per million. Steam-driven plunger pumps in use prior to 1929 were then replaced by Diesel-driven centrifugals; electric drive is now being substituted, because of lower capital cost; suction gas has also been used. In 1927 Babcock and Willcox water-tube boilers were installed, working at 250 pounds pressure and superheat of 190 degrees. In 1930 water pumped was 14,320 million gallons against 130 feet head at coal consumption of 1921 pounds per million gallons. Mileage of mains in 1930 was 170; all larger mains are of steel, unlined, but wrapped, and with minimum cover of 3 feet. All industrial supplies are metered; question of general metering is under consideration, but presents serious domestic problems.—*W. G. Carey.*

Report on the Water Supply of Durban. The Engineer, **152**: 3963, 674–676, December 25, 1931. Present supply is from Umlaas river, where further development impossible; while, as population more than doubled since 1920, enhanced supply is imperative by 1939. Geological characteristics show underground water insufficient, while pumping from rivers in vicinity would cause insuperable difficulties in purification; therefore upland surface supply sought for. Umgeni river, rising in lower slopes of Drakensberg range, was investigated and four schemes suggested. Recommended scheme consists of reservoir holding 5000 million gallons (Imp.), formed by concrete dam 120 feet high and 1300 feet long, and aqueduct 36 miles long delivering into existing service reservoir. Catchment area is 1012 square miles and top water elevation is 1250 feet above sea. This scheme will provide 15 million gallons per day and can be extended to provide at least 50 million gallons per day. It has advantage over others that impounded water would receive prolonged settlement, which has proved to be most essential for economical purification. —*W. G. Carey.*

Corrosion in Water Pipes. L. W. Haase. Zeitschrift fur angewandte Chemie, **44**: 990–992, 1931. Layer of crystalline calcium carbonate protects underlying metal from corrosion, but layer of amorphous calcium carbonate has little protective effect, owing to porosity. Slow stream of water through pipes favors formation of crystalline protective deposit.—*W. G. Carey.*

Colloidal Conditioning of Boiler Feed Water. O. E. Blincow. Journal Institute of Brewing, **37**: 603–607, 1931. Calcium and magnesium salts may be precipitated with colloids, without introduction of sodium salts, in such condition that no scale is formed. Priming and caustic embrittlement are also reduced. Materials may be incorporated with colloid either to neutralize acidity, or to combine with dissolved oxygen.—*W. G. Carey.*

Plumbism. R. L. WILLCOX and E. MILLIGAN. Brit. Medical Journal, 1931, August 1, Pp. 222–3. From Bull. of Hygiene, 6: 11, 817–8, November, 1931. In Glossop, England, infant mortality rate in area with lead-solvent water supply was found to be more than double that in adjacent area where water was treated to prevent plumbosolvency. Lead content of water supply of area with higher rate was nearly ½ grain per gallon [7.2 p.p.m.], or about six times the minimum capable of causing symptoms. Water before entering distribution system is of excellent quality and steps are being taken to prevent its solvent action on lead pipes.—*Arthur P. Miller.*

Purification of Drinking Water by Means of Filters made of Silver-Coated Sand. W. A. UGLOW, A. A. MILLER and KAR-KADINOWSKY. Milit.-Med. Ztschr., Moscow, 1931, 2: 1, 5–11. From Bull. of Hygiene, 6: 11, 822, November, 1931. Experiments have shown that filters of this type not only act mechanically, but also, due to the silver, with bactericidal effect. In one experiment 99.8 percent of applied *B. coli* were destroyed in filters. River or sea-sand, grains of which are from 0.3 to 1.0 millimeter in diameter, is used. Sand is washed, dried, calcined for about two hours, and, while still hot, treated with silver hydroxide. Further details of method of preparing the sand are given.—*Arthur P. Miller.*

On the Photo-dynamic Action of Dyes on Bacteria in the Presence of Phosphorescent Light. H. LINDEN and F. K. T. SCHWARZ. Arch f. Hyg. u. Bakt., 1931, 106: 133–46. From Bull. of Hyg., 6: 11, 832, November, 1931. Study of lethal action of dyes in presence of phosphorescent light on *B. coli* leads to conclusion that such action results from summation of bactericidal actions of dye and of light. Certain dyes are probably taken up by bacterial cells which then absorb light waves which otherwise might have passed through them, or have been reflected from them, and under combined action of dye and of light the organism may succumb. Variations in bactericidal action of same dye on different organims may be accounted for by variation in light-absorbing qualities of the organisms, as well as in primary toxicity of the dye.—*Arthur P. Miller.*

The Dominick-Lauter Method Compared with Standard Methods in Differentiating the Coli-Aërogenes Group in Water Analysis. J. M. MCCANTS. U. S. Nav. M. Bull., 29: 552–9, 1931. From Bull. of Hygiene, 6: 11, 834, November, 1931. Experiments with DOMINICK-LAUTER method of differentiating coli-aërogenes group in water are reported. In accord with results previously given, number of positive presumptive tests confirmed is higher with DOMINICK-LAUTER than with usual standard method. Author believes that this technique may serve to differentiate fecal from non-fecal strains of the coli group, but thinks that further investigational work is needed.—*Arthur P. Miller.*

Position of the Coli Problem. K. VAGEDES. Gesundheits-Ingenieur, 54: 569, 1931. From Water Pollution Research Summary, 5: 3, 90—91, March, 1932. Shape and size of *B. coli* are unreliable for identification. Gram-

negative properties are common to different bacteria; but their absence, in presence of spores and gelatine liquefaction, indicates absence of *B. coli*. Indol formation is practicable method of identification, but its absence is not proof of *B. coli* absence. Fermentation and gas formation are important, but do not differentiate between warm- and cold-blooded origins; latter can generally be distinguished from former by slight, or absent, indol and by absence of gas at 46°C. Virgin soil is free from *B. coli* and it may be assumed that all *B. coli* are of human or animal origin, different forms being due to adaptation. By methyl-red, Voges-Proskauer, and artificial media tests, few in number, but reliable, a large group of this type of bacteria may be differentiated, but finer distinctions, as to types, length of time in water, etc., are unreliable. Serological tests have proved unreliable. Tests on action of bacteriophage, using three different lysines, showed that though these indicated a certain specificity, each acting most strongly on its own source, the action was irregular. The effect of lysines on shape and size of bacteria and on their behaviour in different media, including gas formation and milk coagulation, is discussed. Only properties remaining unaltered are absence of liquefaction of gelatine and decoloration by Gram's stain. Importance of *B. coli* in water supplies is discussed. Taking *B. coli* as sporeless, coccus or rod bacterium, not liquefying gelatine, decomposing dextrose at 37°C. and lactose in Endo agar, presence or absence can be investigated; but more detailed investigation does not lead to greater accuracy. *B. coli* presence in water indicates foreign admixture, source of which should be investigated. Large amount of work in recent years has led to return to simple methods of testing. Possibility of applying knowledge of adaptability of *B. coli* to problems of microbial development and also of action of bacteriophage and of enzyme formation and to colloidal investigations is discussed.—*W. G. Carey.*

Piping a Pumping Station. O. B. Carlisle and D. E. Kennedy. The American City, **46**: 1, 79–83, January, 1932. Piping design at 50th Street and Western Avenue station in Chicago demonstrates latest practice and offers many interesting features and valuable details. There are four centrifugal pumps, each rated at 7 million gallons per day. Each pump is served by vertical concrete shaft extending up from suction tunnel to lower floor level in pump pit, from which point it is connected to pump by cast iron fittings. By means of elaborate header and valving layout, installed in vault built in front of station just below grade, it is possible to serve mains extending along Western Avenue from any one of pumps and to close down any particular service without interfering with the others. Vault is covered in with removable slabs, so that any fitting, or valve, can be replaced from above. Headers are so sectionalized and valved that any pump may discharge to either, or to both, independently of the others. Pressure may be increased on one service main, while maintained normal in the others, by increasing speed of one pump and segregating it to service of main where additional pressure is needed. Sixteen solid-wedge disc type valves, operated by waterproof electric motors acting through cut-tooth gearing, are required to control the discharge system. Eighteen expansion joints are used in the headers and pump discharge piping. These are of sliding type, one part sliding within the other: a bolted ring fol-

lower seals the joint with waterproof hydraulic packing. Both sections of joint and ring follower are lined with ⅛-inch finished copper on the moving surfaces. Thorough anchorage of intervening pipe and fittings rendered expansion joints all the more necessary. Tees, crosses, and laterals of special design were necessary and stresses are based on hydraulic pressure of 186 pounds per square inch, to care for water hammer. Venturi meters, together with water-pressure gages, furnish record of output of station in terms of million-foot-gallons. Condensation on the enormous area of pipe surface was so severe that pipe vault was continuously wet, causing short-circuiting of electrical relay wiring. Entire pipe system in vault was therefore covered with wool-felt insulation and sewed jacket of heavy canvas, thus rendering vault dry and clean.—*Arthur P. Miller.*

Pipe Pit Cuts Cleaning Costs. A. T. CLARK. The American City, **46**: 1, 85–86, January, 1932. When manipulation of chemical balance of water fails to prevent tuberculation, physical removal of tubercules becomes necessary, if pressure and capacity are to be maintained. Cleaner pit developed by New Rochelle Water Company is described, which simplifies cleaning of line of pipe. Pit is at lower end of section to be cleaned and so designed that pipe can be opened therein and water in section discharged without flooding to adjoining culvert. In standard cast iron main passing through cleaning pit, a 12-inch section is made removable and replaceable by being fitted with standard Victaulic grooving and couplings. For cleaning, Victaulic couplings are loosened and short length rolled to the side. Water in line passes out of cleaning pit into near-by creek. At opposite end of 7-mile main is another pit similarly arranged. Between the two, cleaner machine works. Whole section of main can be cleaned in three hours, with maximum elapsed time of five hours, including flow tests before and after cleaning.—*Arthur P. Miller.*

Hydraulic Butterfly Valves. ROSS L. MAHON. Trans. Amer. Soc. Mechanical Engineers, November 30, 1931. This paper describes some of the special features of design found in the modern butterfly valve and which have been adopted within recent years. Means for making this type of valve more effective in limiting leakage are discussed in some detail. Some interesting types of operating mechanisms are outlined, and various kinds of protective and auxiliary equipment are described. Recent applications of butterfly valves for free-discharge service have broadened their useful field, and the suggestion is made that valves of this type might well find future use for other purposes than those existing in the hydraulic power industry.—*A. W. Blohm.*

The Classification of the Colon-aerogenes Group of Bacteria in Relation to Their Habitat and Its Application to the Sanitary Examination of Water Supplies in the Tropics and in Temperate Climates. H. J. O'D. BURKE-GAFFNEY. Jour. Hygiene, **32**: 1, 85, January, 1932. A study of coliform bacteria from various sources, and by means of a number of bacteriological methods, was undertaken in East Africa, with the primary object of ascertaining in what way local standards of water purity might best be established. The methods

employed were the indol test, fermentation reactions of McConkey, methyl-red and citrate tests, Eijkman fermentation test and a direct plating method suggested by the author. A further series of cultures was studied in Europe, with a view to obtaining a comparative estimate of the types of coliform organisms prevailing in these countries. The studies which are given in detail are summarized as follows: (1) the "lactose + indol +" index as employed in temperate climate is usually an adequate criterion of water purity; (2) where a positive test does not appear to be substantiated by the sanitary findings, a further differentiation of the organisms isolated becomes necessary. The methyl-red and citrate tests have been found to supply that differentiation in a satisfactory manner; (3) in the tropics, false positive "lactose-indol" tests are commonly encountered, owing to the presence in large numbers in soil and water of organisms derived from sources other than recently excreted feces, and (4) in the tropics, the lactose-indol test should always be confirmed by the methyl-red, citrate and saccharose tests or such other reactions or groups of reactions as may be found by a local survey to be applicable.—*A. W. Blohm.*

Municipal Engineering for 1930. Surveyor, 79: 2036, 94, January 30, 1931. Water supply. Requirements for water supplies are rapidly changing due to modern sanitation, increasing demand, increasing stream pollution and changing distribution systems. Purity of river waters and ample supplies will be maintained by boards set up to control pollution of watersheds and the distribution of the water therefrom. During 1930 it was made illegal for an engineer to design or officially inspect a reservoir above a certain capacity unless he had been qualified by the government. There is a growing tendency for water districts to be formed to supply an entire county or a group of neighboring towns.—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abstracts*).

Water-borne Typhoid Fever Still a Menace. Abel Wolman and Arthur E. Gorman. Amer. Jour. of Public Health and The Nation's Health, 21: 2, 115, February, 1931. An excellent review of the prevalence of water-borne typhoid and dysentery in the United States and Canada during the decade 1920–1929. Information compiled from reports from 47 states in the United States and from 6 provinces in Canada, representing respectively, 98.65 and 79.5 percent of the populations of these countries. Two hundred and forty-two outbreaks in the United States and 40 in Canada are reported and classified as to source and character of supply, magnitude of outbreak, and causes. Some of the larger outbreaks are discussed in detail. During the decade there were five repeat water-borne outbreaks in one city from the same cause and repeat outbreaks in four from different causes. The courts in both the United States and Canada are increasingly holding cities and water companies liable for heavy financial damages for illness resulting from pollution of public supplies.—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abstracts*).

Bangor (County Down) Municipal Works. Geo. H. Fleming. Surveyor, 79: 2034, 55, January 16, 1931. The water supply is obtained from two catchment areas. The water from the Culig catchment area is stored in two reser-

voirs of 50,000,000 Imp. gallons capacity and that obtained from the Ballysallagh area is stored in a reservoir of 120,000,000 Imp. gallons capacity. The latter system originally included fine sand filters, but owing to the high content of suspended matter the filters clogged quickly and required nearly continuous sand washing. To remedy this condition rapid roughing filters were installed at Ballysallagh to precede the slow sand filters. The roughing filters are washed with air and water and require from 5 to 10 percent of wash water. The high percentage of wash water is probably due to the heavy growth of algae in this catchment area.—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abstracts*).

How a Serious Shortage Can Be Guarded Against. JOHN P. WENTWORTH. Water Works Eng., **83**: 11, 741, May 21, 1930. Attention is called to the extremely low rainfall in the New England States for the past 12 months and that the precipitation was abnormally low during the first part of the year, at the time when reservoirs should have been filling and ground water storage should have been increasing. The possibilities of a serious water shortage are marked and advice is given to authorities to examine carefully the conditions of their supplies and provide for emergency supplies and emergency equipment.—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abstracts*).

Sanitary Regulations of Pools in Schools. H. K. READ. Jour. Amer. Assoc. for Hygiene and Baths, **14**: 17, May, 1930. The swimming pools of the Houston, Texas, public schools are built according to the best known methods. Filtration and chlorination are used as a means of keeping the pool water sterile. The water passes through the filters about four times every ten hours, and a complete change of water is made one time each month. Alum is used as a coagulant and the dosage of chlorine is 1.1 p.p.m. Those using the pools are protected from infection and contagion by strict rules, which are given, governing the use of the pool.—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abstracts*).

Design and Operation of Swimming Pools. A. C. BEYER. Western Construction News, **5**: 20, 527, October 25, 1930. This is a general article on the proper design and operation of swimming pools.—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abstracts*).

Model Bylaws Issued from the Ministry of Health. VIII. Public Bathing. Ministry of Health, 1929. London: H. M. S. O. Bulletin of Hygiene, **5**: 3, 219, March, 1930. To enable local authorities in Urban Districts to control bathing along seashores or strands, Model Bylaws have been framed as guides to administrative action.—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abstracts*).

Protecting New Jersey Bathing Beaches. Public Works, **61**: 7, 84, July, 1930. Practically the entire of the New Jersey shore line for more than 100 miles is a bathing beach where 40 municipalities maintain beaches. Along the southern part of the shore, sewage is disposed in inland waters, but from Beach Haven north it is discharged into the ocean. The New Jersey State

Department of Health requires seashore municipalities discharging into the ocean to provide at least sedimentation and chlorination of the sewage and the use of outfall sewers extending not less than 1,000 feet into the ocean. Dye is added to the treated sewage during investigations to indicate whether the outfall sewer is leaking. This test also furnishes data as to the path of travel of the sewage in the ocean. Current investigations have indicated that the effluent travels either up or down the beach in a very narrow stream and that enormous quantities of ocean water rapidly dilute and permit oxidation of the effluent. It also appears that the surface film of polluted water reaches the beach, but only after it has been so highly diluted that it is of better bacterial quality than surf water where bathing is active.—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abstracts*).

Cook County Provides Safe Swimming. CHARLES G. SAUERS. American City, **44**: 1, 139, January, 1931. Cermac Pool, a modern swimming plant in Cook County, Illinois, is 100 by 250 feet in plan and has a capacity of 850,000 gallons. The pool accommodates 25,000 at one time without undue crowding. Non-slip floors eliminate many accidents. Rigid inspection and a soap and water bath in the nude is required of all bathers. The water purification plant includes two electrically driven centrifugal pumps, double sterilizers, ample hair strainers, pressure filters of the horizontal type, filled with machine cut quartz, flow indicators and suction cleaner.—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abstracts*).

Sterilisation of Drinking Water by Iodine. Le Genie Civil, 634, June 28, 1930. Water and Water Engineering, **32**: 381, 452, September 20, 1930. The iodine method is stated to be as efficacious as the chlorine method and by far easier in application; with iodine, grossly polluted water can be perfectly sterilised. Notes are included of tests undertaken in Chile and France, and some results of iodine treatment given. Effect of tabloids is described; and this war time prescription, found satisfactory in France, is reproduced: Iodine 30 cgrm., iodide of potassium 60 cgrm., water 60 cc. Six drops of this solution were added per liter of water to be sterilised.—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abstracts*).

The Wolman and Weaver Formula for the Calculation of the Number of Coli-germs in Water and its Derivation. N. D. R. SCHAAFSMA. Reprint from the Mededeeligen van den Dienst der Volksgezondheid in Ned.-Indie, Part I, 1930. Water and Water Engineering, **32**: 382, 497, October 20, 1930. "Describes tests carried out at Manggarai Water Purification Laboratory in the Dutch East Indies. McCrady's simplified formula for calculation of number of coli-germs in water has been used for several years at the State Bureau for Water Supply in Holland, this being modified by Wolman and Weaver's insertion for interpretation of fermentation tube results. In checking the derivation of McCrady's expression an error was found in the log. signs. In the mathematical exposition it is shown that Napier's log. signs must be substituted for those used in the McCrady formula. The following summary of conclusions is given: (*a*) derivation of McCrady's formula for the calculation

of the most probable number of coli-germs in a given quantity of water is verified; (*b*) a general formula is computed for cases in which series of tests are prepared of quantities of water whose number of cubic centimetres is not a whole positive or negative power of 10; (*c*) it appears that such series in which all tests are either positive or negative may be considered non-existent."—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abstracts*).

De-chlorination Treatment by Activated Charcoal. L'Eau, 76, May, 1930. Water and Water Engineering, 32: 382, 497, October 20, 1930. "Procedure is detailed of the method employed by Dr. R. Cambier, director of the Paris Laboratory of Hygiene. For elimination of taste and odour from chlorinated water, Dr. Cambier considers the physical process by activated charcoal to be the best. This charcoal—of vegetable origin—has by special treatment been given a strongly developed capillary structure so that water passing through a granular bed is relieved of every trace of free chlorine as well as of colouring and other objectionable matter. Experiments have shown that for treatment of water containing 1 mgm. to 1.5 mgm. of chlorine one bed of charcoal has sufficed to purify samples for over a year. With a small apparatus, water containing 1 mgm. of chlorine per liter can be thoroughly treated at the rate of 1 kgm. of charcoal for a constant delivery of 1 cm. of water per day; with this amount, B. coli and phenol products are removed. Dr. Cambier describes types of apparatus suitable for domestic and municipal purposes and states that the small cylinder type filled with charcoal can be fixed by soldering to mains and will be quite effectual against excess chlorine dosage."—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abstracts*).

New System of Aeration Developed for Water Use. Water Works Engineering, 83: 11, 852, May 21, 1930. The new system is that as patented by Vogt Bros. Mfg. Company, called Aero-mix. Instead of finely dividing the water by spraying or cascading, the air is finely divided through air tubes located in a manifold and broken up into very fine particles. The air and water pass down a down draft then into a riser containing spiral baffles. The air and gases pass into the atmosphere and the water is directed to its normal channel. There are no moving parts, the entire process being based on the air being drawn into the mixing chamber and then broken up into very fine particles.—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abstracts*).

Maine Association Issues Cross Connection Report. Water Works Engineering, 83: 11, 808 and 812, May 21, 1930. The Main Water Utilities Association presented a report and recommended that the association approve the regulations and present them to the Public Health Council of the State of Maine for adoption. The rules and regulations were to govern the installation and operation of cross connections and provided that after a certain date no new cross connections or additions to existing ones will be permitted, and that after a certain date all existing cross connections shall be eliminated except where protected by approved devices, installed and inspected under the supervision of the State Health Department. The duty of reporting installations of cross connections would rest with the firm or corporation maintain-

ing such connection. Double check valves and removal sections of pipe are approved devices. All connections would be provided with automatic chlorinating devices. Penalties of from $100 to $10,000 are provided for first and second offenses.—*A. W. Blohm (Courtesy U. S. P. H. Eng. Abstracts).*

Citizens Privately Finance New Water Works for Bloomington, Illinois. L. R. Howson. The American City, **43**: 5, 131, November, 1930. During 1929 and 1930 Bloomington, Illinois, constructed an impounding reservoir and a modern purification and softening plant to replace the unsatisfactory deep well supply which had been used by the city for about 50 years. The new project was financed privately by twenty-five leading citizens subscribing to one share of stock each, making a total of $2,500. With this as a nucleus, the construction cost of $1,200,000 was financed through the sale of $900,000 in bonds and $300,000 in preferred stock.—*A. W. Blohm (Courtesy U. S. P. H. Eng. Abstracts).*

Explanation of North Carolina State Board of Health, Standard Report. R. D. Beam. Jour. North Carolina Section A. W. W. A., **7**: 1, 13, 1929. The standard daily and monthly water purification plant reports, furnished by the State Board of Health to the filter plants is explained in detail with instructions as to how to fill out. The value of complete records and accurate cost data in convenient form and properly filed are emphasized.—*A. W. Blohm (Courtesy U. S. P. H. Eng. Abstracts).*

Ground Water or Surface Water? Hayo Bruns. Vom Wasser, 4, 82. The total water consumption in Germany is estimated to be for 1929 about 2,500 million cubic meters, of this about 77 percent is ground water, 14 per cent surface water and about 8 percent artesian well water. Of the ground water about one-fourth is obtained by infiltration galleries. The surface water is about half filtered stream water, one-third stored water and the rest is sea water. The water consumption has increased greatly during the last 30 years. Thirty years ago the water consumption was from 15 to 25 gallons per capita per day and at present from 40 to 50 gallons per capita per day. Of the total 2,500 million gallons used in Germany about one-third is used in the Rhineland-Westphalia industrial center, a good deal of this is collected in infiltration galleries. The water price for industries is already high and if it is taken in consideration that in the Ruhr District 70, 80 and sometimes 90 percent of the water is used by the industries it is apparent that extensive treatment of surface water is impossible.—*A. W. Blohm (Courtesy U. S. P. H. Eng. Abstracts).*

Purifying Water for the Textile Industry. S. B. Applebaum. Water Works & Sewerage, **79**: 83–6, 1932. Description of installations for water purification in a cotton finishing plant, a carpet factory involving the scouring and dyeing of wool, and a rayon factory, respectively.—*C. C. Ruchhoft (Courtesy Chem. Abst.).*

Method of Handling and Control of Chlorine in Ton Containers. M. C. Smith. Water Works and Sewerage, **79**: 63–4, 1932. Chlorine in ton con-

tainers is cheaper and can be handled more conveniently. Description is given of method of handling tanks and of control in chlorine room.—*C. C. Ruchhoft.*

Filter Sand Studies. JOHN R. BAYLIS. Water Works & Sewerage, **79**: 43–46, 1932. Studies on filtering materials are being made by American Society of Civil Engineers and American Water Works Association. Best and most generally used method for grading filter sand is that developed by HAZEN. Need is emphasized for more study on filtering materials and filter runs, to aid the two societies in their work.—*C. C. Ruchhoft.*

Super- and Dechlorination at Glencoe, Ill. E. L. PFLANZ. Water Works & Sewerage, **79**: 93, 1932. Raw water receives chlorine dosage of 1 to 1.5 p.p.m., to maintain residual of 0.5 p.p.m. Contact period is about 6 hours, through mixing basin, coagulation basin, and filters. Sodium bisulfite is used to neutralize excess chlorine. Water free from tastes and odors is produced.—*C. C. Ruchhoft* (*Courtesy Chem. Abst.*).

New Water Softening Plant at Neodesha, Kansas. F. M. VEATCH. Water Works & Sewerage, **79**: 75–6, 1932. Supply is taken from Fall River, which in spring and summer has turbidity of from 200 to 5000 p.p.m. and hardness of from 100 to 200 p.p.m., while, in fall and winter, turbidity is low and hardness reaches 350 p.p.m. Softening plant treats 2 m.g.d. Provision has been made for use of alum, lime, soda ash, chlorine, ammonium sulfate, and carbon dioxide. A hopper-bottomed, combined mixing and aëration tank is used. Settling tank holds a 12-hour flow. Stilling baffle has been inserted in first compartment and second compartment has been baffled to provide recarbonation chamber, mixing channel for alum, and final settling basin. Filter plant consists of two hydraulically operated filter units, each of 1 m.g.d. capacity. Carbon dioxide is obtained by combustion of natural gas. Dry-feed machines are used. Chlorine is applied into filter effluent line just before it enters clear well. Cost of bringing hardness down to average of 100 p.p.m. and of producing clear, tasteless, odorless, and bacterially safe water is 1.25 cents per 1,000 gallons, exclusive of fixed charges.—*C. C. Ruchhoft* (*Courtesy Chem. Abst.*).

Hints in the Coagulation of Water. JOHN R. BAYLIS. Water Works and Sewerage, **79**: 71–74, 1932. Method of judging coagulated water is to collect sample from effluent of mixing basin. If most of flocculated matter settles in 5 to 10 minutes, it is properly coagulated. If water is so cloudy after 10 minutes settling that particles of flocculation farthest from the eye cannot be detected, or are only faintly visible, it very likely will not filter clear. Observation either of water in settling basins, or of appearance of floc in path of beam of light thrown into basin, is also of aid in determining good coagulation. Solution feeds, chemical feeding boxes and chemical feed lines are sources of trouble due to wearing away of parts and clogging of lines. Dry-feed machines are more satisfactory. Improved coagulation due to agitation can be obtained by inserting boards about 8 inches wide and 10 to 12 feet long flatways across

line of flow in first section of mixing basin. Correct pH is essential for proper coagulation.—*C. C. Ruchhoft (Courtesy Chem. Abst.).*

Purifying Water for the Textile Industry. S. B. APPLEBAUM. Water Works & Sewerage, 79: 47–51, 1932. Soft water is necessary for production of soft, clean, odorless, and uniformly dyed wool. Zeolite softening has resulted in saving of about 50 percent in soap and of more than 25 percent in ammoniated potash. Use of soft water in boiling-off, bleaching, dyeing, and mercerizing of cotton is important. If water is hard, a curd will deposit upon cotton fibre which will prevent successful bleaching, mercerizing and dyeing. Similar precautions are necessary in manufacture of silk and of rayon. Ideal water for textile purposes should be free from turbidity; low in color, organic matter, iron, manganese and alum; and of zero hardness. Clarification process must be carefully worked out. Zeolite softening is found very satisfactory in textile industry.—*C. C. Ruchhoft (Courtesy Chem. Abst.).*

Right of City to Pollute Water. LEO T. PARKER. Municipal Sanitation, 2: 10, 472. October, 1931. Municipality situated on arm of sea, adjacent to tidal waters, has right to use such waters for sewage disposal so long as public nuisance is not created. There is considerable distinction between polluting (*a*) small non-navigable stream and (*b*) large tidal navigable body of salt water. In first case, bed and waters of stream have riparian owners; in latter, both are owned and controlled by state, for use and benefit of all the public, subject only to navigation. It is for state to say what uses shall be made of navigable streams and by whom, subject always to right of the public. Also, state may say how much pollution will be permitted into and upon its waters, so long as owners of land between low and high water mark are not injured. **Oyster Beds.** LOVEJOY *vs.* City of Norwalk, 152 Atl. 210. Statutes provided that committee should designate suitable place to be used by applicants for planting oysters and that every person who should plant oysters in any place so designated should be and continue the owner of such oysters so planted. City of Norwalk drained sewage into Norwalk River. Oyster grounds are situated under the navigable water in Norwalk harbor and below the low-water mark, about two miles off shore. The owner filed suit to recover damages caused by city sewage disposal. Higher court concluded that pollution was confined to tidal waters and did not constitute public nuisance and that exercise thereof by the city was not in derogation of any right of owner of the beds. Hampton *vs.* WATSON, 89 S.E. 81. Action was filed against city to recover damage to oyster bed by pollution of waters of Hampton Creek, a tidal navigable body of salt water, an arm of the sea, by sewers emptying therein. The owner legally operated. Authorities found that Hampton Creek was too polluted to permit the sale of oysters therefrom. Owner sued the city. Higher court held the city not liable. **Complaining Party Must Prove Water Pollution.** MILLER *vs.* City of Woodburn, 294 Pac. 349. Property owner sued to enjoin city from emptying sewage into small creek which flows through his land. Owner also complained that natural flow was increased to his damage. It was shown that sewage from greater number of homes was disposed of by septic tanks which had no connection with city system. Stream afforded drainage for numerous

barnyards, a cannery, a slaughter house, and a cemetery; which would tend to pollute the water independently of city sewage system. Court refused party owner damages. **Municipality Liable for Violation of Court Order.** Department of Health of State of New Jersey *vs.* Borough of Ft. Lee, 154 Atl. 319. Certain property owners filed suit to enjoin municipality from discharging polluted materials from sewer system into small creek. Lower court enjoined city from polluting the water. Municipality did not abide by court order. Owners appealed to higher court, which held city liable for payment of fine of $10.00 a day for violation of injunction.—*R. E. Noble.*

NEW BOOKS

Laboratoriumsbuch für die kolorimetrische Wasseruntersuchung. PAUL MARTINY. Halle: W. Knapp. 73 pp. M. 5.40; bound, M. 6.80. Reviewed in Z. angew. Chem., **44**: 916, 1931. From Chem. Abst., **26**: 1051. February 20, 1932.—*R. E. Thompson.*

Electric Arc Welding. Troy, Ohio: Hobart Bros. Co. 80 pp. From Chem. Abst., **26**: 1561, March 20, 1932.—*R. E. Thompson.*

Manual of Dehydrated Culture Media and Reagents. 3rd edition, revised. Detroit: Difco Laboratories, Bacteriological Division of Digestive Ferments Co. 166 pp. From Chem. Abst., **26**: 1636, March 20, 1932.—*R. E. Thompson.*

L'Eau que l'on boit à Paris. G. LEMARCHAND. Paris: Dunod. 178 pp. F. 40. From Chem. Abst., **26**: 1690, March 20, 1932.—*R. E. Thompson.*

Das Wasser in der Natur und im Dienste des Menschen. HANS HEINZE. Freiburg: Herder & Co. 164 pp. M. 3.60. Reviewed in Nature, **129**: 223, 1932. From Chem. Abst., **26**: 2000, April 10, 1932.—*R. E. Thompson.*

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The Design of Dams. FRANK W. HANNA and ROBERT C. KENNEDY. New York and London: McGraw-Hill Book Co. Inc. Cloth: 6 x 9 in.; pp. 456. $5. Reviewed in Eng. News-Rec., 108: 260, February 18, 1932.—*R. E. Thompson.*

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cal, or semi-tropical, health officer. Author has stressed the application to the tropics of principles applied to selection and conservation of water and conditions essential to preserving its purity.—*Arthur P. Miller.*

The Government of Metropolitan Areas in the United States. PAUL STUDENSKI with the assistance of the Committee on Metropolitan Government of the National Municipal League. New York: National Municipal League, 261 Broadway. Cloth; 6 x 9 inch; pp. 403. $3.50. Reviewed in Eng. News-Record, 106: 659, April 16, 1931.—*R. E. Thompson.*

National Aspects of Water Power Development: A Review of the Facts. National Water Policies Committee. Washington, D. C.: Chamber of Commerce of United States. Paper; 8 x 10 inches. $1. Reviewed in Eng. News-Record, 106: 660, April 16, 1931.—*R. E. Thompson*

Land Drainage and Flood Protection. BERNARD A. ETCHEVERRY. New York and London: McGraw-Hill Book Co., Inc. Cloth; 6 x 9 inches; pp. 327. $3. Reviewed in Eng. News-Rec., 106: 903, May 28, 1931.—*R. E. Thompson.*

Modern Diesel Engine Practice: A Comprehensive Treatise for the Student and Practical Engineer. ORVILLE ADAMS. New York: The Norman W. Henley Publishing Co. Cloth; 6 x 10 inches; pp. 656. $6. Reviewed in Eng. News-Rec., 106: 903, May 28, 1931.—*R. E. Thompson.*

Colorado River and the Boulder Canyon Project: Historical and Physical Facts in Connection with the Colorado River and Lower Basin Development. Colorado River Commission of the State of California, F. H. MCIVER, Secretary. Sacramento: Colorado River Commission. Paper; 6 x 9 inches; pp. 400. Reviewed in Eng. News-Rec., 106: 1022, June 18, 1931.—*R. E. Thompson.*

Powdered Activated Carbon for the Removal of Tastes and Odors from Water. Pamphlet of Industrial Chemical Sales Company, N. Y. I. CHARLES E. TROWBRIDGE. First plant experiments with powdered carbon were made at South Pittsburgh in November, 1930, when phenol and other substances were present in water due to drought. Carbon was applied to untreated water because feeding points giving surety of thorough distribution of carbon were available. At times 135 pounds of carbon per million gallons were used to remove tastes and odors. At present, experiments in ten plants show better results, using 5 to 20 pounds per million gallons added ahead of filter. No material reduction takes place in length of filter runs. Theory is that particles of 200-mesh carbon diffused through water offer enormous absorption surface. Application through ordinary water injector by dry feed provides the moisture essential for thorough distribution. Principal advantage is that objectionable substances are removed and not merely decomposed. Result is higher and more uniform quality of water throughout distribution system. Concentration by distillation is applied to samples of filter effluent in testing for taste- or odor-producing substances. Powdered activated carbon is a valuable addition to methods of purification. II. E. F. DUGGER. Water supply for

city of Newport News is derived from shallow lakes. Algae produced tastes and odors and caused many complaints, especially during drought of 1930 when growth was heavy. Following the drought, water took on higher color and pronounced taste and odor. Steps were promptly taken to apply powdered activated carbon. From barrel, equipped with agitator in form of electric hand drill with wooden paddles attached and with control valves, carbon suspension at rate of 5 pounds of Nuchar No. 2 per hour was fed into filter influent about 25 feet from first filter. Treatment did not shorten filter runs. Cost was 75 cents per million gallons. Eight hours after treatment begun, filter effluent was free from all tastes and odors and complaints ceased. **III.** R. W. FITZGERALD. Norfolk, Va., utilizing water of Lake Prince, was faced with three-fold problem in simultaneous presence of (1) vegetable material in solution; (2) high putrescible organic matter in sludge, imparting bad tastes in settling basins; and (3) seasonal algae growth, producing tastes varying from musty and earthy to slightly fishy. Prechlorination was not altogether successful. Experiments proved that ammonia-chlorine treatment would not destroy algae tastes. In 1931 taste trouble became so acute that trial run using Nuchar powdered activated carbon was started at both plants. Within a few hours, practically tasteless effluents were produced. Complaints ceased entirely. For experimental purposes, use of carbon was stopped at both plants: within two days complaints of bad tastes were numerous. Carbon treatment was resumed, with practical cessation of complaints. In October, usual lake turn-over occurred; one complaint was received, but condition was quickly relieved by increasing carbon, average dose being less than $\frac{1}{10}$ grain per gallon. Cost was approximately 70 cents per million gallons. **IV.** J. S. WHITENER. One cubic inch of carbon has an approximate surface area of 20,000 square yards. Specifications of Nuchar activated carbon are: 94 per cent passing 200-mesh sieve; 90 to 95 percent pure carbon; 5 to 10 percent ash, of which two-thirds is soluble in hydrochloric acid; water extract varies from neutral to slightly alkaline, and weight from 10 to 15 pounds per cubic foot. Apply with dry feed machine with ejector on solution hopper. Best point to apply is in effluent of coagulation basin, to give thorough mixture before filtration: other points are: entrance mixing chamber; entrance, or middle, of coagulation basin, giving longer contact, sludge stabilization, and increased mass action in colored water. **Results** from using activated carbon in eight different plants are given. **Conclusions** (1) Tastes and odors from algae and decomposing organic matter can be eliminated. (2) Prechlorination tastes are eliminated. (3) Carbon treatment will most probably be much cheaper than aëration. (4) Sludge in coagulation basins is stabilized. (5) Tests show that carbon never penetrated filter beds beyond 4 inches. (6) Negligible reduction in filter runs. (7) Cost is from $0.35 to $1.30 per million gallons. (8) Carbon need only be applied when tastes and odors are present; but some plants use it continuously as safety factor. (9) Only chemical effect of carbon is slight change in pH.—*R. E. Noble.*

Proceedings of the 14th Texas Water Works Short School, Mineral Wells, Texas, January 18–20, 1932. 187 pages. **Section I. Water. Our Program for Bettering Water Supplies in Texas.** J. C. ADAMS. 17–19. State of scope

of services offered by State Health Department to water supply plants. **Relation of City Health Department to Water Supplies.** A. H. FLICKWIR, W. N. DASHIELL. 19–23. City Health Department exercises supervisory control of public, private, domestic and bottled water supplies under its jurisdiction. **Modern Methods of Aëration.** HERMAN B. ANDERSON. 23–27. Discussion of advantages gained by combined aëration and mixing, as secured by the Aer-O-Mix. Operating results from use of Aer-O-Mix at Waukegan, Illinois; Lawton, Oklahoma; Oshkosh, Wisconsin; and Lynchburg, Virginia. **Iron Removal without Filtration, Kansas State Penintentiary.** E. T. ARCHER. 27–30. Description of improvements to water supply and state institutions at Lansing, Kansas. Aëration and lime treatment followed by improvements in existing sedimentation basins reduced iron content of water from 7 to 0.2 p.p.m. **Facts Affecting Corrosion of Delivery and Service Mains.** R. H. FASH. 31–32. Discussion of electrochemical theory of corrosion which holds that iron in contact with water goes into solution, replacing hydrogen which collects on unattacked surface of the iron in thin film. If hydrogen is not removed, solution of iron thereupon ceases, but if hydrogen be removed, either by combination with oxygen, or as bubbles of hydrogen gas, corrosion continues and results in generation of electric current, the anode at point of corrosion and the cathode at unattacked surface of the iron. Study of corrosion of pipe lines by U. S. Bureau of Standards and others has determined that electric currents flow along pipe lines leaving the line and entering the soil where there is sufficient soil moisture to afford an electrolyte and soil resistance is low. Survey of soil resistivity along pipe line routes provides index of points where corrosion may be active and special protection should be provided. **The Ammonia-Chlorine Treatment of Water and its Recent Application at Wichita Falls, Texas.** A. H. DOUGAAS, 32–36. Chloramines are products formed by interaction of ammonia and chlorine in such proportions that either one or two hydrogen atoms of ammonia molecule are replaced by chlorine atoms. Chemical reactions which take place in ammonia-chlorine treatment of water are determined by kind and relationship of elements naturally present in water. Calcium and magnesium may form hypochlorites which may undergo secondary reaction with ammonium hydroxide to form calcium and magnesium hydroxides and ammonium hypochlorite. Active sterilizing agent of chloramines is chlorine. Difference in action between chlorine and chloramines is due to slowing down of disinfection efficiency of chlorine. Ammonia is now proving a very convenient vehicle for application of chlorine in water works practice in that it admits of use of higher rates of chlorine feed than would be practicable with chlorine alone. **Ammonia-Chlorine Treatment of Water.** JULIAN MONTGOMERY. 37–40. Study of recent experiences of many cities with ammonia-chlorine treatment indicates that in majority of cases taste elimination is successful and that in all cases it is partly so, the degree depending upon the nature of water supply and skill with which treatment is applied. Ammonia should be applied prior to filtration only in conjunction with prechlorination. There is no definite ratio of ammonia to chlorine most effective in every case. Contact period of two hours should be provided when water temperature is low and pH value high. **Relative Usefulness of Well Water Pumps; Centrifugal Turbine Air Lift, and Piston Types.** J. G. MONT-

GOMERY. 40–44. Centrifugal turbines have efficiencies from 70 to 85 percent; air lift, from 20 to 35 percent; and piston pumps from 40 to 75 percent. Air-lift offers greatest delivery for given area of well casing. It is the only pump that may be practically used in casings that are out of plumb. **Demonstration and Discussion of Methods of Determining Efficiencies of Centrifugal Pumping Units.** M. L. WHITNEY. 44–48. List of instruments for measurement of head and quantity of water and of speed of pump, directions for establishing operating conditions during test, formulas and conversion factors for computing results. **The Use of Activated Carbon in Water Purification.** E. L. FILBY. 48–55. Activated carbon is valuable for removal of organic taste and odor producing compounds, for positive dechlorination, for retardation of sludge decomposition, and for assistance in color removal and coagulation. For taste and odor removal it must be used as an adsorption unit, either in granular or in powdered form. Its dechlorination capacity permits high concentrations of chlorine to be applied, as for heavy bacterial loads, and later removed, without danger of chlorinous tastes. In cases where offensive odors are given off by settling tank sludges, these can be reduced by treatment of incoming water with activated carbon. In certain cases activated carbon has been found beneficial in absorbing flat tastes of softened, but not carbonated, water, in removal of color, and in assisting flocculation of clear well waters. **Taste and Odor Control in Drinking Water Supplies.** L. C. BILLINGS. 55–62. Coagulation, addition of chemicals, aëration, and adsorption with activated carbon are effective means of taste and odor elimination from water. In plants where coagulation and sedimentation are important processes of treatment, a large portion of taste-producing organic material is removed by scrubbing action of coagulating particles. Addition of ammonia with chlorine permits more intensive application of chlorine without producing chlorinous tastes. Superchlorination and dechlorination are effective through direct oxidation of organic material. Potassium permanganate also oxidizes organic material, but care must be taken against overdosing with this chemical. Aëration is effective through mechanical sweeping out of occluded gases and by oxidation. Adsorption with activated carbon is applicable with all known organic taste-producing compounds. **Discussion.** W. T. Gooch. 63–64. Washing of idle filters just previous to return to use has been found effective in eliminatin taste and odor from this source. **Water Works Valuation in Rate Making.** K. D. FOWLER. 64–70. Value of property includes both physical value and going-concern value. Physical value includes direct costs of labor, materials, and equipment plus general costs of engineering, insurance during construction period, and profit, fairly reduced by an amount for physical depreciation (accidents and wear and tear) and for functional depreciation (obsolescence, inadequacy, and supersession). Going-concern value includes preliminary expenses of engineering surveys, legal services and organization expense, construction period expenses of interest on capital, taxes, and management costs and development period expenses covering operating losses during the growth of business, or its market, to paying volume. **Some of the Results of the Coöperative Federal and State Survey of the Underground Waters of Texas.** W. N. WHITE. 70–76. U. S. Geological Survey in cooperation with State Board of Water Engineers investigates underground waters of Texas and issues

from time to time reports of results, covering available yields of strata, waste of water, contamination by soft water, decline of artesian heads. Long records of draft and fluctuations in heads of ground-waters are invaluable in these studies. **Determining Factors in the Selection of Industrial Water Purification Equipment.** O. A. DECELLE. 76–80. Complexity of the many problems of water treatment, especially for industries, demand unbiased study and well founded recommendations for each specific problem. More important problems of water purification and practical means of treatment available are tabulated. **Pollution Hazards of Ground-Water Supplies.** F. H. PUCKHABER. 80–89. The general belief in purity and sterility of underground water is unwarranted, as is shown by records of many epidemics traceable to contamination of wells. Liability of water supply corporation, whether private, or municipal, for unsafe water has been established and reaffirmed by the courts. **Criteria for Determining the Sanitary Quality of a Drinking Water Supply.** CHESTER COHEN. 89–92. The bacterial standard alone is not a fair and safe criterion for drinking waters from all sources. For surface waters, sanitary survey should be considered in interpreting results of bacterial analyses. For shallow well supplies, knowledge of local conditions is essential. For deep well supplies, geological formation through which well is drilled and construction of well are principal indices of quality. Deep well waters should not be judged as to sanitary quality by the presence of aërogenes organisms. "The present propaganda for universal chlorination of well waters, despite their origin, cites evidence of epidemics traceable to well waters; but the epidemics were not caused by well failures, per se, but rather by permitting sewer lines, deep cesspools, and abandoned unplugged wells to remain adjacent to the wells." **Discussion.** J. W. BASS. 92–94. Most certain guarantee of pure water supply should be sought in unquestionably pure source, rather than in subsequent purification. **Boiler Feed Water Problems.** HELMAN ROSENTHAL. 94–99. Impurities in water are suspended solids of siliceous, or clay, materials, dissolved solids of various rock minerals, and gases nitrogen, oxygen, and carbon dioxide. Carbonates and bicarbonates of calcium and magnesium form scale. Sulphates and chlorides of calcium and magnesium form scale and may be corrosive at high temperature. Carbonates and bicarbonates of sodium may case priming, foaming, and corrosion. Mineral acids, oxygen, and carbon dioxide are causes of corrosion. The common forms of chemical treatment for boiler feed water are discussed and theories of priming, foaming, and embrittlement are cited. **Discussion.** C. M. BARDWELL. 99–102. Foaming is caused by combination of soluble salts and suspended matter. Most practical method of control is by blowing down. Pitting is caused by dissolved gases or acid ions in solution, involving replacement of hydrogen ion in water. By maintaining sufficient concentration of caustic alkalinity, hydrogen ions are nearly eliminated and rate of corrosion is greatly reduced. Embrittlement is due to caustic soda and is reduced in direct proportion to amount of sodium sulphate present. **What Constitutes a Satisfactory Water for Ice Manufacture.** J. C. TUCKER. 104. Water for ice manufacture must have fairly low content of dissolved solids. Processes for rendering natural waters acceptable for this purpose include chemical treatment, air agitation, and core removal. Treatment with lime, calcium hypochlorite,

magnesium zeolites, alum, or sulphuric acid may be suitable, according to specific impurities present. **Service Installations.** ART WOODS. 104–109. Corrosive soil conditions in El Paso led to study of suitability of various pipe materials which concluded with pronouncement in favor of copper service pipe. Because of consumers' difficulty in operating curb cocks in case of need, to shut off house supplies, compression cocks are now installed instead. **Installation and Repair of Water Mains.** W. H. WARING. 109–114. Practical discussion on water main construction including handling of pipe, excavation, laying of pipe, location and setting of fittings, valves and hydrants, joint materials, back-filling, routine inspection, and emergency repairs. **Pipe Jointing Compounds and Materials.** D. W. ROBINSON. 114–119. *Poured lead joints.* Advantages: general familiarity with materials and methods, ease of testing, repairing, or salvaging, good conductivity for electrical thawing, adaptability for submarine use. Disadvantages: requirement for large bell-holes, high cost, no elasticity, lack of resistance to temperature changes, or vibration, difficulty and danger of calking in caving trench, failure to tighten up if lead starts. (2) *Lead wool joints.* High cost; but particularly useful for submarine work. (3) *Precalked lead joints.* Advantages: unnecessary to yarn, or pour, joints in field. Disadvantages: similar to poured lead joints. (4) *Lead substitutes.* Advantages: low cost, ability to withstand vibration and temperature changes, tendency of leaky joints to take up, lower conductivity decreases electrolysis, less danger of blowouts or explosions in wet trenches due to lower pouring temperature. Disadvantages: uncertainty in unskilled hands; prejudice of employees trained with poured lead; greater time required to carry out test; difficulty of repairing leaks and salvaging pipe. (5) *Portland cement.* Advantages: low cost, ability to withstand temperature changes and vibrations, tendency of leaky joints to take up, lower conductivity reduces electrolysis. Disadvantages: requirement for large bell-holes; requirement for experienced calkers; prejudice of older employees; difficulty of testing and repairing. **Equipment Necessary for the Efficient Operation of a Small Water Works Service and Repair Shop.** T. L. LAIN. 119. **Mineral Water Production and Purity of Bottled Water.** W. J. MILES. 120–126. The stronger mineral waters of Mineral Wells, Texas, carry about 360 grains per gallon of solids, of which over 300 grains per gallon are sodium sulphate. State Board of Health and City Health Department for the bottling companies maintain strict sanitary control over quality of water sold. **Section 2—Sewage. Recent Developments in Sewage Disposal.** CLYDE C. HAYS, B. A. BASSEL. 127–131. **Sludge Digestion.** H. E. SCHLENZ. 133–142. **Discussion.** P. J. ALVIN ZELLER. 143–146. **Improved Methods for Sewer Installation.** J. W. PORTER. 146–151. **Municipal Sanitary Status in Texas February 29, 1932.** 152–180. Tabulation showing sources of water supply, water treatment, and sewage disposal of municipalities of Texas.—*Robert L. McNamee.*